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Chapter 1 Handbook Introduction

1.1 Purpose of Handbook

The purpose of the Electronic Field Book Processing (EFBP) handbook is to be a guide for the use of that software system. EFBP is one component of Geopak Survey. The intent is not to present the theoretical background for EFBP, as that information is in a companion technical document. Many surveyors may consider both documents necessary when using EFBP.

If one is using EFBP in conjunction with a particular Electronic Field Book (EFB) survey data collection system, one is strongly urged to contact sources for references on use of that EFB. Successful use of EFBP is enhanced when one has a strong understanding of the system which was used to collect the survey field data.

EFBP is the engine for numerical processing of raw survey data (horizontal circle readings, zenith circle readings, slope distances, height of instrument, height of target, level rod readings, taping, calibrations, point names) into coordinate information. EFBP does not process attribute information (feature codes, zones, remarks/comments, chains/linework, straight vs. curve line geometry, etc.) but carries this information into ascii readable format so that it can be read with the coordinate information into the rest of Geopak. The use of EFBP is often termed "processing" one's data.

This document assumes a user has some land surveying background, has access to common land surveying textbooks as this document is not intended to eliminate the need for that information, and that a user has some fundamental personal computer knowledge specifically with MS-DOS background.

This document makes extensive reference to files using the MS-DOS naming convention. EFB and EFBP files usually consist of a consistent project name before the period, and a 1-3 character extension which alerts the user what the file is. This document makes reference to files using only the period and the extension (.ctl as an example), while it is assumed that the project name always exists before the period.

1.2 Discussion of components of the handbook

While an initial user will find reading of this information in a systematic start-to-finish fashion beneficial, users of EFBP will often be using the table of contents to move quickly to information that is immediately desired.

The first discussion of EFBP is with specific regard to its use with the Florida Department of Transportation's Electronic Field Book (EFB) survey data collection software. EFB has similarity to other survey data collection systems which EFBP supports, and thus users who bring data into EFBP from other collection systems will still find this discussion useful. EFBP reads an observation (.obs) file for field information and a control (.ctl) for control coordinate information. The initial discussion will make constant reference to information which is standard format in the .obs file. The format of this file will be clearly addressed in chapter 4.

Chapter 5 discusses management of control coordinate information using computer program CTL. CTL has a wide variety of options for making control coordinate management as efficient a system as possible. CTL options are being incorporated into Geopak Survey's control point manager.

Chapter 6 discusses the option menu in EFBP. EFBP is a non-interactive batch process once the desired options are accepted in the initial menu. Chapter 7 introduces the components of EFBP. Chapters 8, 9, and 10 explain in detail the three output reports from EFBP (abstracting, 1D least squares, and 2D least squares). Chapter 11 discusses the final coordinate production process and subsequent final file creation necessary for import to other software systems. Chapter 12 discusses the use of four utility programs included in the EFBP system.

Chapter 13 is a discussion of the basic concepts of least squares analysis with emphasis on error estimation of one's measurements. Chapter 14 is a basic discussion of state plane coordinate systems and their impact on EFBP and other software systems.

Chapter 15 is an extensive series of examples of how the EFBP reports are used in troubleshooting the survey data and recognizing what needs to be corrected. Chapter 16 details a general strategy for data processing, project storage, and program storage. Chapter 17 is a description of executables, required system data files, and input/output files for EFBP.

Chapter 2. EFB as it pertains to processing.

2.1 Basics of use of EFB

EFB is a series of command menus and data screens that allow the surveyor to electronically describe the survey field collection process. With the correct interface serial cable it is able to automatically record data from a total station or an electronic level. It also allows manual input of these numerical values at any time.

To use EFB effectively, one must be aware of how EFBP and the rest of Geopak use the data. EFB does not replace the need for a knowledgeable field surveyor, but instead that person is the most critical element in making the office system more efficient.

2.2 Why station naming is so important

To successfully utilize EFBP, the concept of station (point) naming must be fully understood. The rule is quite simple: once a station is named, that name must always be used in any subsequent measurement to it or instrument occupation of that point. This rule relates to both sideshots and traverse (redundant) stations. This rule is required because EFBP is trying to assign one unique set of coordinates to that point name. Multiple coordinates for the same point name are not allowed.

EFB utilizes a prefix/suffix point naming convention. The prefix can be alphanumeric though common practice is to keep it all letters. The first and last characters of the prefix are not allowed to be numeric when using EFB. The suffix is always numeric and is assigned by EFB or by the surveyor. The combined length of the prefix and suffix cannot exceed 8 characters, and the combination of the two components, with no space or other character in between, defines the point name. Names such as A10, TREE1, TREE1012, CONTROL9 can exist. EFB does not allow a prefix to end with a number.

EFBP allows up to eight alphanumeric characters for a point name. Unlike EFB, the name can be totally numeric or totally alpha characters. There is no need for any numeric values in the point names. If numeric names or suffixes are used, there is no need for sequential order, and gaps in the numbering are allowed. Thus station names like 1, 1012, COE, or DAC are permitted. A job can have point numbers 1 to 110, then a gap, and continue from 320 to 378. EFBP treats upper and lower case letters as different, thus station name COE is not equivalent to station name Coe. While EFBP accepts a station name with a space in it, it is not recommended.

Examples of station naming problems twofold:

(1) The same physical point gets assigned two different names. An example of this is a sideshot to a rebar which represents evidence of a property corner which was measured from two traverse stations. If from one setup it was called C1 and from another setup it was called C2 after EFBP is completed you will have slightly different coordinates for C1 and C2. If you named it C1 from both setups EFBP will use all the measurements to it in a least squares solution for the statistically "best" coordinates for the station.

Many data collectors and office survey software systems force one to give a new point name when closing a loop traverse to its initial point. This would create no identification of the closure to EFBP. Thus when closing to that point it is given its initial point name.

(2) Two different physical points get assigned the same name. An example would be where A23 was assigned to a traverse station on the northerly edge of the job, and by mistake a station on the southerly edge of the job was also referred to as A23. You have told EFBP these two unique stations are the same point, and thus EFBP's reports will show you some very poor closures.

EFBP does not use the point name by itself to figure out if the point is a traverse or sideshot station. As an example two different trees were assigned point name TREE12. EFBP does not look at the name and figure out that trees are usually never measured twice. EFBP does use the name to recognize that TREE12 was measured twice and is thus a redundant point. The user would look at the report files, see that TREE12 is in the least squares report, and realize one of those shots needs to be assigned a different point name.

EFBP (and EFB) do not dictate how you perform your survey. Many software systems force you to adopt their way of performing the survey. EFBP and EFB do force you to use the point naming logic of "one point name per one field survey point" as that is the key to EFBP recognizing how the survey was performed.

2.3 Use of Reference Names

EFB has a unique method of control station naming that EFBP's control station manager computer program CTL can take advantage of. Note one does not have to use the reference name at all if desired. A reference name is a second name which can be assigned to a point which designates that the point is an existing control station, or that the coordinates generated for it may be used as control in a later part of the survey. A reference name can be up to 16 alphanumeric characters, and is not prefix/suffix in nature like EFB station names.

An example of use of a reference name is assigning its organizational name for it such as BASS_1963. Organizations could be the National Geodetic Survey, Department of Transportation, Army Corps of Engineers, other government agencies, private control surveys, etc. The reason for it can go beyond simply documenting the official name of the point. It allows multiple crews working on the same job to assign different field names (prefix/suffix as discussed in 2.1) to control points, and CTL can assign control coordinates to those points via the reference name. Examples of use of reference names are in chapter 5.

If reference names are not used, the field survey names are used in linking coordinates in the control file (.ctl) to measurements in the .obs file. Thus a user can use EFBP without ever considering use of reference names.

2.4 Types of Records

2.4.1 Header

The header record in an EFB collected .obs file contains the units of measure (feet or meters), the segment name, and optional remarks about the project. The header is not used by EFBP. EFBP reads units from the control .ctl file. The units in the header are used in the field by EFB in correctly storing distances which are electronically derived from the total station. The header is at the top of the .obs file, and only one header exists per .obs file. EFBP will process in absence of a header record.

2.4.2 Calibration

The calibration is also not required for successful use of EFBP. EFB does force storage of a calibration record. Most of the calibration record is information not used by EFBP such as temperature, atmospheric pressure, weather code, observer's initials, notekeeper's initials, rodperson's initials, instrument brand and model, instrument serial number, EDM absolute error, EDM ppm error, stadia constant, level test indicator, axis test indicator, and comments indicator.

Many users of EFBP have thought it corrects distances for temperature and pressure. This is not true! You should be dialing those corrections into your instrument, and EFB will then be storing values that are corrected for temperature and pressure systematic errors. Different instrument manufacturers use different error models for the correction, and thus it is better that the correction is performed at the time of data collection.

Some users have also mistakenly thought EFBP uses the EDM constant and ppm errors in the calibration for error estimation in the least squares analysis. Again this is not true as that information is defined in the initial options menu of EFBP.

Calibration allows you to perform a peg test on the level, or a horizontal and vertical axis test on a total station. The peg test is not numerically used by EFBP as the user is supposed to adjust the level's cross hair based on the test results. The total station axis test contains a user defined number of direct and reverse pointings on the same well defined point. EFBP determines operator pointing error, and systematic errors in the vertical and horizontal circles. The systematic errors are corrected in all subsequent measurements in the .obs file. EFBP can operate with no calibration. If multiple calibrations exist in a .obs file the calibration derived systematic error corrections are applied to all subsequent measurements until the next calibration is reached. The new values are used until the next calibration is reached, etc. If multiple unique calibrations are stored in immediate succession the last one is used in correction of the subsequent measurement information.

2.4.3 Setup

The instrument setup record is used by EFBP to obtain occupied station name and height of instrument. Height of instrument is irrelevant if 2-D survey observations are made or if differential leveling is being performed.

Other attribute type information that can be entered about the setup is geometry (point/straight or curve), attribute, zone, comments, feature code, and reference name. This information is not used in numerical processing (EFBP) but becomes critical when the data enters a subsequent graphics system. Reference names are used in managing control coordinates in CTL and are not used directly by EFBP.

2.4.4 Observation

Observation components can be divided into the numerical measurements and user defined values associated with them, and the attribute information associated with the point that has been measured.

2.4.4.1 Numerical measurements

EFB allows for three types of measurements:

(1) total station type (HVD)
(2) station/offset/rod reading (SOR)
(3) taping

2.4.4.1.1 HVD and derivatives of it

H, V, and D refer to horizontal circle, zenith (vertical) circle when zero is assumed pointed up, and distance (usually slope) respectively. The user can collect HVD, HD, HV, H, VD, or D as options. Note HVD and HV (if no prism on backsight) are the most common types of measurements being made. Associated with D is a S (slope), H (horizontal), or V (vertical) label with S being predominant.

Also associated with numerical processing are:

(1) station name

(2) height of target - measured vertically from ground point to the prism. Note this is unimportant for a 2-D survey, but measuring target and instrument heights for all setups is a recommended procedure.

(3) eccentricity - If a point cannot be directly occupied an offset horizontal distance from the object to the prism can be entered and the prism defined as F (forward), B (back), L (left), or R (right) of the object. On left/right eccentrics a right angle is assumed at the prism between the line to the instrument and the line to the object. EFBP automatically uses eccentric information to obtain coordinates of the object.

(4) Telescope Orientation - This is labelled as D (direct) or R (reverse). D assumes zenith circle readings between 0 and 180 degrees, while R assumes zenith circle readings between 180 and 360 degrees. This is critical when EFBP reduces horizontal circle readings to horizontal angles, and in applying calibration corrections. H mode requires a D or R on a shot (the only time this needs to be hand entered as EFB recognizes it from the zenith circle) as EFBP needs to know what orientation the instrument was in for correct processing. If a zenith angle is being recorded EFB will store the correct D (or R) in .obs .

(5) Position set number - In turning angle repetitions, it is very common for one to average horizontal angles and not circle readings. This procedure is critical if the horizontal circle is advanced between angle repetitions. The position set number is usually toggled whenever the backsight is re-shot and a series of observations are going to be made based upon it. The position number is generally not toggled between direct and reverse pointings though EFBP does not mind either practice. EFBP does treat direct and reverse pointings on the same position set as unique measurements because it is assumed calibration corrections can be applied which correct for the reasons for doing direct and reverse readings. In the past direct and reverse readings were often averaged before computing repetition error because instrument calibration was not known.

Other attribute type information that can be entered about the HVD type observation is geometry (point/straight or curve), attribute, zone, comments, feature code, and reference name. This information is not used in numerical processing (EFBP) but becomes critical when the data enters a subsequent graphics system. Note reference name is used in managing control coordinates in CTL and is not used directly by EFBP.

2.4.4.1.2 SOR and derivatives of it

Station and offset (SO) is an economical procedure for efficient location of survey points relative to a defined center or base line which is called a geometry chain. R means differential rod reading and can be single or three wire readings. Two wire readings can be entered but only the middle wire reading is used in numerical processing. You have the options of SOR, SO (horizontal location only), or R (differential leveling only)in this mode. As usual the station name is used in numerical processing.

Other attribute type information that can be entered about the SOR type observation is geometry (point/straight or curve), attribute, zone, comments, feature code, route (geometry chain), and reference name. This information is not used in numerical processing (EFBP) but becomes critical when the data enters a subsequent graphics system. Note reference name is used in managing control coordinates in CTL and is not used directly by EFBP.

EFBP does not convert the station-offset information to horizontal coordinates because in many situations the entire geometry chain/baseline does not yet have required information such as curve parameters. Subsequent functions in Geopak are able to process the station-offset information into coordinate form. The station-offset data is transferred by EFBP to a station-offset-elevation (.soe) file that is readable by functions in Geopak.

2.4.4.1.3 Taping

Taping is a procedure which assumes right angles and horizontal distances. This form of surveying is very useful when only horizontal positions of topographic features are required, those positions are not required to be of high accuracy, and the process can eliminate time consuming total station instrument setups. In the field the surveyor defines occupied, backsight, and foresight station names, then inputs a horizontal distance from the occupied to the foresight station along with a F (forward), B (back), L (left), or R (right) orientation designation. The orientation is at the occupied while looking at the backsight.
This procedure is very useful in locating back building corners once two corners of the building have been located with the total station.

Taping exists in the .obs file as H (backsight) and HD (foresight) records and follows all of the other survey measurements in the file. It is processed as if it were made with the total station.

Other attribute type information that can be entered about the taping type observation is geometry (point/straight or curve), attribute, zone, comments, feature code, and reference name. This information is not used in numerical processing (EFBP) but becomes critical when the data enters a subsequent graphics system.

2.4.4.2 Measurement Attributes

While some of the information attached to measurement has no importance in numerical processing, these types of information are defined here as they relate specifically to EFB.

(1) Geometry - This is with respect to the point being part of a chain (line). The point is on a portion of the chain that is a straight line - Point (P), or on a portion of the chain that is part of a curve (C).

(2) Attribute - This is a subclassification relative to a surface (usually ground) and can be Ground (G), Feature (F), Cross section (X), or User defined (U). A G implies the point should be part of a digital terrain model (DTM), while a F indicates it should not be part of the DTM. Cross section is a more descriptive type of ground attribute. A U is up to user definition as to what it means.

(3) Zones - This is a way of segregating data in a database sense. Zones can be used to distinguish different surfaces or different types of utilities. Zone is an integer number.

(4) Feature code - This is a designation of what the point is. EFB allows up to 8 alphanumeric characters. This is usually linked to some symbology library when viewing data. Note this is a point feature code, which is different from a chain feature code. Many points that are parts of chains (edge of pavement) require no point symbology and thus receive no point feature code, but are part of a chain which has a chain feature code and thus links to line type symbology.

(5) Comments - These are any extended textual information about the point which was measured which is beyond the definition of the feature code. If TREE was being used as a feature code, the comment may include tree type, diameter, crown diameter, etc.

2.4.5 Chains

Chains are linework. Note the point information indicates whether the points are on a straight or curved portion of a chain. Chains have no numerical processing needs, but are provided here as defined in EFB. A chain can include:

(1) Chain name - Chain names work under the same prefix/suffix logic as point names. Names are required for each chain.

(2) Attribute - The same ground G, feature F, or user defined U options as existed in point attributes exist here. A G attribute indicates to Geopak that the chain is a break line in a DTM.

(3) Zone - This has the same meaning as in point information. It is a way to segregate data such as surface models.

(4) Feature code - This has the same meaning as in point feature codes except that they relate to lines.

(5) Comments - This is for more extensive describing of a chain beyond a feature code.

(6) Station - Stationing can be associated with the chain mainly for definition of a cross section.

(7) List - This is a sequential list of the point names in the chain. A comma exists between point names. If two commas exist in sequence, this is a location where no line is to be drawn ("pen up"). FDOT allows for "macro" defining of chains such that:
A1-5 is the same as A1, A2, A3, A4, A5.
B12-10,Q12,A6-8,,9-10 is the same as B12, B11, B10, Q12, A6, A7, A8,, A9, A10.
A means A1, A2, A3, etc.

Chapter 3. Field files and conversion of them to ascii input (.obs) to EFBP.

The measurement input to EFBP is in a file with an .obs extension. If data is collected with FDOT's EFB the .obs file is constructed from 5 field files.

EFB field data consists of 5 files. All have the same DOS name (project name and segment usually) and have different extensions which define what the files are:
(1) .RAW - a binary file of measurement (HVD and SOR) information
(2) .PRE - an ascii file designating used point prefixes and their last suffixes
(3) .CHN - a binary file of chain information
(4) .CPX - an ascii file designating used chain prefixes and their last suffixes
(5) .TAP - a binary file of taping measurement information

These files need to be downloaded from the data collector to a directory on your PC.

To convert these files to the .OBS file required for processing, FDOT has a public domain program called TSMTOASC which is accessed by Geopak Survey. Note all programs have a .exe extension to their name. That program should be in the same directory as your data or you should have a path to it.

To convert to .OBS be sure you are in the directory where your five files exist. At the DOS prompt type TSMTOASC and press enter. The program will prompt you for your DOS project name which exists for all of your field files. Type in this name and press enter and the five files will be converted to an .obs file. Any problems that occur during the translation will be shown to you on the computer screen. Many of the messages that appear on the screen are warnings and can often be ignored. As examples, warnings of calibration without data or a setup without measurements usually mean in the former the surveyor decided an instrument calibration was not necessary at that time, and in the latter it usually means the "real" setup is stored immediately after the one with no data. Neither of those warnings adversely affects processing. When the program is terminated you should be able to view the .obs file in a text editor (an example is EDIT which comes with recent versions of DOS) and print it.

If you are using SDMS (Survey Data Management System) for survey data collection Geopak Survey provides an option which converts that file type (usually a .prj extension) to .obs . All other data collector translations are handled by Geopak through the G&G download followed by conversion to .obs .

Chapter 4. Structure and explanation of the observation (.obs) file

The .obs file is very structured in nature. Editing of it to correct any problems needs to be made cautiously as the file is read by EFBP and other systems as "fixed format". This means it is assumed certain columns contain certain types of data. For station names and other alphanumeric files left justification to the proper column is required. For real numbers the column of the decimal point is critical. For integers (degrees and minutes) left justification to the proper column is required. Thus the rule is place changes back in their same column locations.

To help one with the placement of data in columns, in this chapter every segment of .obs file is preceded by:

```
123456789112345678921234567893123456789412345678951234567896123456789712345
```

to illustrate the proper columns.

You will also notice the first four columns of a row of .obs designate what the line represents. Column two is usually blank. If a D is in column 2 it means this line is marked as deleted and processing ignores it. M in column 2 means modified - that line is again ignored as it is the old information that was not desired and immediately adjacent to it is the new information which should be read.

All angular values are in degrees (integer) minutes (integer) seconds (real) format with space(es) between degrees, minutes, and seconds.

4.1 Header Record

An example of a header record is:

```
123456789112345678921234567893123456789412345678951234567896123456789712345
H 00                        17:00:57 08/12/95  17:00:57 08/12/95 ENGLISH
```

The time and dates refer to project activation time. The ENGLISH would be replaced by METRIC if the linear information in the file was of that unit type.

4.2 Calibration Record

An example of a calibration record is:

```
123456789112345678921234567893123456789412345678951234567896123456789712345
C 00 17:02:34 08/12/95  88 30.0 001101 NBW JDO BLP JM
C 01 TOPCON     GTS-3B/3C               5  3
C 03 17:06:22                       D   0  0  0.0  90  9  4.0
C 03 17:06:32                       D   0  0  1.0  90  9  7.0
```

```
C 03 17:07:07                      R 179 59 41.0 269 51  3.0
C 03 17:07:25                      R 179 59 41.0 269 50 58.0
```

The C 00 line contains time, date, temperature, pressure, weather code, and field crew initials in that order. The C 01 record contains brand and model of the instrument, EDM constant error in millimeters, and EDM ppm error. The C 03 records are the pointings for a total station calibration and from left to right include time of observation, direct or reverse, horizontal circle, and zenith circle.

4.3 Setup record

An example of a setup record is:

```
12345678911234567892123456789312345678941234567895123456789612345678971234 5
S 00 AA1                       P F1 12
S 01 20:28:57 08/12/95  5.570
```

What follows the S 00 is station name, geometry, attribute, zone, and feature code. The feature code may have a dash followed by a description. The S 01 record contains time, date, and height of instrument.

4.4 Observation record

```
123456789112345678921234567893123456789412345678951234567896123456789712345
O 00 AA2                       P F1 13
O 01 20:30:11    5.270           1 D   0  0  0.0  90 12 39.0   353.830 S
O 01 20:30:40    5.270           1 R 179 59 52.0 269 47 21.0   353.830 S
O 00 BM1                       P F1 13
O 05 08:53:05    5.000           1 R             269 35  0.0    49.530 S
OM01 20:31:36    5.000           1 R  18 29 58.0 269 35  0.0    49.530 S
O 05 08:53:15    5.000           1 D              90 24 51.0    49.560 S
OM01 20:32:38    5.000           1 D 198 31  2.0  90 24 51.0    49.560 S
O 00 EPR64                     P G3
O 99 ON NORTH EDGE OF COE ROAD BY DRIVEWAY
O 01 20:35:31    5.000           1 D 150 57 33.0  89 48 59.0   204.740 S
O 00 CD67                      P G3
O 01 20:36:25    5.000           1 D 147 39 46.0  89 47 54.0   211.460 S
```

The O 00 record is independent of data type and includes station name, geometry, attribute, zone, and feature code. The feature code may again have a dash followed by a description. The O 01 means HVD data.
Other data types are 02 - HD, 03 - HV, 04 - H, 05 - VD, 06 - D, 07 - SOR, 08 - SO, and 09 - R. In HD and D records the distance can only be horizontal as no zenith angle is associated with it. Note for derivatives of HVD as evidenced by the 05 type in the example, the data not measured is left blank. If someone changes an 01 to an 04 record the zenith angle and slope distance will be ignored even if they exist in that line.
In the above example BM1 refers to a benchmark and it was desired to only measure elevation change to it so the H component was removed.

The O 01 record contains time, height of target, position number, telescope orientation (D or R), horizontal circle, zenith circle, and distance (usually slope) for HVD data. A comment about the point is a O 99 record, and it exists between the O 00 and the O 0x record(s) where x refers to the observation type (1-9). Note all measurements to a point from a setup are immediately below that station's O 00 record. In the case of more than one measurement to a point at a setup the data is thus not necessarily in the order in which it was collected. Having all measurements to a point dramatically enhances examination of the data for potential problems. From the example it can be seen from this setup AA2 was measured twice.

The next section of observation data illustrates a point measured 3 times each in direct and reverse position where each D/R combination was assigned a new position number (immediately to the left of the D/R designation). One can see the horizontal circle was reset between position numbers. This also shows the location of a reference name (55-17-A16) that has been entered for a field survey name (FCI1).

```
123456789112345678921234567893123456789412345678951234567896123456789712345
O 00 FCI1     55-17-A16       P G1
O 01 14:33:48    5.850          1 D 359 59 59.0  89  8 51.0  4203.670 S
O 01 14:34:37    5.850          1 R 179 59 56.0 270 51 22.0  4203.660 S
O 01 14:50:04    5.850          2 D 162 50 54.0  89  8 42.0  4203.680 S
O 01 14:59:26    5.850          2 R 342 51  8.0 270 51 28.0  4203.630 S
O 01 15:00:39    5.850          3 R 250 54 54.0 270 51 23.0  4203.670 S
O 01 15:10:03    5.850          3 D  70 54 51.0  89  8 44.0  4203.670 S
```

The next section of observations reflect two examples of eccentrics. The first is 0.76 ft. left and the second is 0.34 ft. forward.

```
123456789112345678921234567893123456789412345678951234567896123456789712345
O 00 FL14                     P G1
O 01 20:30:17    1.700   0.76 L  1 D 347 27 19.0  89 44 58.0   105.680 S
O 00 UP2                      P G1 22-P,CTV,XFMR
O 01 19:51:42    1.500   0.34 F  1 D 329 29 27.0  89 57 11.0    61.899 S
```

The next series of data illustrates SOR type data. The observation to B7 illustrates 3 wire leveling, the second shows an example where only two wires could be read, and the third example is single wire differential leveling. Note differential leveling (R) is record type O 09. The observation to D1 is SO (record type O 08). 1025.00 is station 10+25.00 and the offset is 75.50 ft. right (positive offset is right). The geometry chain/baseline is identified as XS1. The last example is SOR (record type O 07) and illustrates a left (negative) offset.

```
123456789112345678921234567893123456789412345678951234567896123456789712345678
O 00 B7       TP1             P G1
O 09 14:14:13                                     6.185 6.051  5.920
O 00 B2       KOGER-8         P G1 17
O 09 19:01:29                                     1.372 0.531
O 00 B1       KOGER-5         P G1 17
O 09 23:19:16                                           7.05
O 00 D1                       P G1 50              XS1
```

```
O 08 15:12:50     1025.00   75.50
O 00 D18                       P G1 24                         XS1
O 07 16:42:34     1046.20  -61.29                                    4.250
```

4.5 Taping record

An example of a taping record follows. Note taping always is immediately after the HVD and/or SOR data.

```
1234567891123456789212345678931234567894123456789512345678961234567897123450
R 00 00:00:00 12/31/99
R 99 TAPE OBSERVATION DATA STARTS HERE
C 00 00:00:00 12/31/99
C 99 DUMMY TAPE CALIBRATION TO RE-INITIALIZE COLLIMATION TO ZERO
C 01 TAPING     TAPING              10 99 100
C 03 00:00:00                     D   0  0  0.0  90  0  0.0
C 03 00:00:00                     D   0  0  0.0  90  0  0.0
C 03 00:00:00                     R 180  0  0.0 270  0  0.0
C 03 00:00:00                     R 180  0  0.0 270  0  0.0
S 00 CD99                     P F1
S 01 18:48:35 08/13/22  0.000
O 00 CD98                     P F1
O 04 18:48:35    0.000          1 D   0  0  0.0
O 00 CD100                    P F3
O 02 18:48:35    0.000          1 D 180  0  0.0                20.000 H
S 00 BLD2                     P F1
S 01 21:03:30 08/20/22  0.000
O 00 BLD1                     P F1
O 04 21:03:30    0.000          1 D   0  0  0.0
O 00 BLD3                     P F3
O 02 21:03:30    0.000          1 D 270  0  0.0                18.500 H
R 00 00:00:00 12/31/99
R 99 TAPE OBSERVATION DATA ENDS HERE
```

The standardized remark records (R 00 and R 99) define the start and end of taping records. Taping is simply presented in .obs as an H record on the backsight and an HD record on the foresight. The horizontal circle reading on the backsight is always zero, and on the foresight it can be zero, 90, 180, or 270 degrees. Taping is always preceded by a "dummy" calibration record ensuring no systematic error correction occurs as that only applies to measurements by the total station.

4.6 Tape Traverse

After the taping observations are the tape traverses/chains. This shows the order in which a series of taping occurred. It is very useful in finding blunders (usually errors in forward, back, left, right designations) in the taping processes. Standardized remarks identify the start and end of the tape traverse data.

```
1234567891123456789212345678931234567894123456789512345678961234567897123450
```

```
R 00 00:00:00 12/31/99
R 99 TAPE TRAVERSE DATA (I.E. CHAINS) START HERE
F 00 TAP                            TAP
F 01 BLD17,BLD18,BLD19,BLD20,BLD21,BLD22,BLD23,BLD24
F 00 TAP                            TAP
F 01 FENCE17,FENCE16,FENCE18
R 00 00:00:00 12/31/99
R 99 TAPE TRAVERSE DATA (I.E. CHAINS) ENDS HERE
```

4.7 Chains

After the tape traverses are the chains. Standardized remarks identify the start and end of this type of information. A particular chain consists of three types of records:

(1) F 00 - This line includes chain name, attribute, zone, and feature code.

(2) F 01 - The chain point list as it was entered in the field.

(3) F 02 - The chain point list in non-abbreviated form. The ",," (pen up) field record is replaced by the word GAP.

F 01 and F 02 records may occupy more than one line as they never extend beyond column 72 and a line always ends in a comma between station names in the list.

```
R 00 00:00:00 12/31/99
R 99 CHAIN DATA STARTS HERE
F 00 CLP3                        G3 109
F 01 CLP35-37,,38-40
F 02 CLP35,CLP36,CLP37,GAP,CLP38,CLP39,CLP40
F 00 BL33                        G6 111
F 01 BL310-314,,315-317,,318-319,,320-324
F 02 BL310,BL311,BL312,BL313,BL314,GAP,BL315,BL316,BL317,GAP,BL318,BL319,
F 02 GAP,BL320,BL321,BL322,BL323,BL324
R 00 00:00:00 12/31/99
R 99 CHAIN DATA ENDS HERE
```

4.8 Prefix record

The last record in the .obs file is the prefix record. It is identified by standardized remarks (R 99 records). The P 01 records are the point prefixes which have been used followed by the largest numeric suffix which was used for that particular prefix. The P 02 records are the chain prefixes which have been used followed by the largest numeric suffix which has been used for that particular prefix. These records are generated directly from the field .pre and .cpx files when TSMTOASC is executed.

```
123456789112345678921234567893123456789412345678951234567896123456789712345
R 00 00:00:00 12/31/99
```

```
R 99 PREFIX DATA STARTS HERE
P 00
P 01 A22,AA4,AD6,AP4,BCL67,BCR129,BL324,BLL13,BLR27,BM4,CB37,CC10,CD110,
P 01 CFP11,CI50,CLP44,CP5,CR58,CW31,DBL4,DBLL4,EPL32,EPR85,FENCE18,FH4,
P 01 GL40,GR66,GRASS50,GV1,HW5,LP2,MB12,MH15,PIPE52,PL1,PP28,R208,SAN6,
P 01 SC3,SHRUB94,SIGN18,STREET4,SU4,SW11,SWL58,SWLL24,SWR80,SWRR30,TCL60,
P 01 TCR108,TDL5,TDLL6,TH26,TL1,TP4,TPED2,TREE130,TVP1,UGG7,UGT8,UGW7,
P 02 AD1,BCL6,BCR11,BL33,BLL3,BLR3,CB2,CD7,CFP1,CI9,CL1,CLP4,CP1,CR1,CW3
R 00 00:00:00 12/31/99
R 99 PREFIX DATA ENDS HERE
```

Chapter 5. Control point management (program CTL)

5.1 Program requirements

CTL is the control coordinate manager for EFBP. The control file has the extension .ctl, and has a very standardized fixed format. If a control file does not exist for a project, EFBP will assume coordinates of 10000 N, 10000 E, 500 Z (elevation) for the first setup and an azimuth of north to the first measurement at that setup. Note if your data is not totally connected by measurements, i.e., control coordinates are required to generate non-connected segments of data, you will not produce coordinates on all of your points without a .ctl file. Likewise if your first setup contains no data and was not deleted, or if your first setup is leveling and HVD data exists later, coordinate generation will not be possible without a .ctl file. Note in these cases you can still process for abstracting/repetition error information (.gen) only as that is independent of coordinates.

EFBP uses field survey names in identifying points. EFB and CTL lets you use reference names for points but their use is not required.

CTL is executed by typing CTL and pressing enter at the DOS prompt. If a TEMP.JOB file exists the project name is read from that file, and you have the ability to change that project name. If TEMP.JOB does not exist, you will be asked to enter a project name.

CTL reads geodetic information such as datum year, state plane zone identification, coordinate units, vertical datum year, and type of azimuths from the .ctl file. When switching projects if a .ctl does not exist for the new project name, the geodetic information from the previous project is retained as the defaults.

CTL is case sensitive (upper or lower case) with regards to control station names and how they match to station names in the .obs file. The best suggestion is have CAPS LOCK activated during all EFBP processing. Note a station name of BASS1 in .obs will not match to a station name of Bass1 in .ctl. The match of names in .obs and .ctl is imperative.

CTL has a large series of menus. A discussion of these menus and their input will follow. User input will be shown in [ ] though note only what is inside the [ ] occur as input. Items in { } are comments describing the input process for discussion in this manual. Thus while running a program no { } or anything inside of them would appear.

5.2 Input of Datum Parameters

{If TEMP.JOB did not exist you are asked to input a project name. If it did exist the project in that file is the default and the user can change it.}

```
COMPUTER PROGRAM CTL VERSION 06-14-94

INPUT PROJECT NAME

[COE]

CONTROL DEFAULTS (ASSUMED INDICATED BY NO INFORMATION)
(1) STATE PLANE NGS ZONE ID:
(2) HORIZONTAL DATUM YEAR:
(3) UNITS:
(4) VERTICAL DATUM YEAR:
(5) AZIMUTH TYPE:
(6) ACCEPT DEFAULTS
PICK THE # OF THE DEFAULT TO CHANGE <6>

[1]  {Select the NGS state plane zone #.  If this job is in assumed
     coordinates no input of any of these parameters is required so
     one could immediately select 6.}

INPUT STATE PLANE NGS ZONE ID (ENTER IF NO CHANGE)
ENTER A ZERO IF ASSUMED OR A QUESTION MARK FOR HELP

[0901] {This is the Florida East Mercator zone number.  Entering a ? lets
       you view the list of state plane zones with their numbers.}

CONTROL DEFAULTS (ASSUMED INDICATED BY NO INFORMATION)
(1) STATE PLANE NGS ZONE ID: 0901
(2) HORIZONTAL DATUM YEAR: 27
(3) UNITS:
(4) VERTICAL DATUM YEAR:
(5) AZIMUTH TYPE:
(6) ACCEPT DEFAULTS
PICK THE # OF THE DEFAULT TO CHANGE <6>

[2] {Once a zone is selected the program defaults to NAD 27.  Option 2
     is selected so a user can select another horizontal datum.}

INPUT HORIZONTAL DATUM YEAR (ENTER IF NO CHANGE)

[83] {NAD 83 is selected.  Note one can input other years to designate an
     adjustment year, such as for a HARN.  Input greater than 82 uses the
      NAD 83 ellipsoid information.  Input less than 83 uses the NAD 27
     ellipsoid information.}

CONTROL DEFAULTS (ASSUMED INDICATED BY NO INFORMATION)
(1) STATE PLANE NGS ZONE ID: 0901
(2) HORIZONTAL DATUM YEAR: 83
(3) UNITS: F
(4) VERTICAL DATUM YEAR:
(5) AZIMUTH TYPE:
(6) ACCEPT DEFAULTS
```

PICK THE # OF THE DEFAULT TO CHANGE <6>

[3]

INPUT F FOR SURVEY FT, I FOR INTERNATIONAL FT, M FOR METERS <F>

[M] {The program defaults to survey foot for units. The user input M to switch to metric units.}

CONTROL DEFAULTS (ASSUMED INDICATED BY NO INFORMATION)
(1) STATE PLANE NGS ZONE ID: 0901
(2) HORIZONTAL DATUM YEAR: 83
(3) UNITS: M
(4) VERTICAL DATUM YEAR:
(5) AZIMUTH TYPE:
(6) ACCEPT DEFAULTS
PICK THE # OF THE DEFAULT TO CHANGE <6>

[4]

INPUT VERTICAL DATUM YEAR (ENTER IF NO CHANGE)

[29] {While no reductions are necessary for elevation differences such as scale and elevation factors, it is important to designate which datum your elevations are referenced to. The standards are 29 (NGVD 1929) and 88 (NAVD 1988) though other years could designate some other regional elevation adjustment.

CONTROL DEFAULTS (ASSUMED INDICATED BY NO INFORMATION)
(1) STATE PLANE NGS ZONE ID: 0901
(2) HORIZONTAL DATUM YEAR: 83
(3) UNITS: M
(4) VERTICAL DATUM YEAR: 29
(5) AZIMUTH TYPE:
(6) ACCEPT DEFAULTS
PICK THE # OF THE DEFAULT TO CHANGE <6>

[5]

INPUT A FOR ASTRONOMIC, G FOR GRID < >

[G] {If you have no azimuths in your control file, no input is required here. All azimuths have to be grid or all astronomic (geodetic). Usually the difference between astronomic and geodetic north is insignificant. If it is significant, astronomic values should be converted to their forward geodetic equivalents before entry. There is no need to input azimuths between two horizontal control points as their coordinates define that value.}

CONTROL DEFAULTS (ASSUMED INDICATED BY NO INFORMATION)
(1) STATE PLANE NGS ZONE ID: 0901

(2) HORIZONTAL DATUM YEAR: 83
(3) UNITS: M
(4) VERTICAL DATUM YEAR: 29
(5) AZIMUTH TYPE: G
(6) ACCEPT DEFAULTS
PICK THE # OF THE DEFAULT TO CHANGE <6>

[] {At this point you have entered all your geodetic default parameters. When you run CTL again after control has been entered these values will automatically appear as the defaults. At any point you can correct them but any latitude/longitude input may have been incorrectly changed to state plane in the wrong datum, zone, or units. Likewise changing units here does not change the existing coordinates - this is an option in a later menu.}

{Note the number in < > is the default and pressing enter [] produces the same results. This concludes geodetic default maintenance and now the main menu for control maintenance appears.

5.3 Input of control coordinates and azimuths

1-INPUT BY KEYBOARD /DELETE CONTROL POINTS
2-INPUT/DELETE AZIMUTHS
3-LIST DATA
4-CHANGE (EDIT) EXISTING DATA
5-IMPORT CONTROL FROM .XYZ OR .CTL FILES
6-FREE FORM COORDINATE INPUT
7-CHANGE UNITS (FT., M., INTER. FT.)
8-CHANGE STATE PLANE ZONE
9-IMPORT BY REF. NAME
10-QUIT
CHOOSE A # <10>

[1] {Input some control coordinates}

DO YOU WANT TO?
    1-ADD DATA
    2-DELETE DATA
    3-QUIT
INPUT THE # OF YOUR CHOICE: <3>

[1]

ENTERING A "U" FOR UNDO ALLOWS ONE TO ELIMINATE PREVIOUS ENTRIES

{During control entry an input of U backs you up to the previous input so you can re-type it due to a keyboard entry mistake. You can back up a limited number of multiple inputs.

ARE INPUT COORDINATES
(1) LATITUDE , LONGITUDE

```
(2) NORTHING (Y), EASTING (X)
PICK A # <2>

[1] {example of latitude/longitude entry}

INPUT CONTROL STATION NAME: (ENTER IF FINISHED)

[A1]

ENTER CONTROL STATION REFERENCE NAME (ENTER IF NONE)

[]

INPUT LATITUDE IN DDD.MMSSSSS FORMAT (ENTER IF BENCHMARK ONLY)

[28.540239478]

INPUT LONGITUDE IN DDD.MMSSSSS FORMAT

[81.231094554]

STATE PLANE COORDINATES [automatic conversion geodetic to state plane]
NORTHING (Y) =   506082.330  EASTING (X) =   162318.288
INPUT CONTROL ELEVATION (Z) (ENTER IF NONE)

[9.234]

INPUT CONTROL STATION NAME: (ENTER IF FINISHED)

[BM1] {enter another control station}

ENTER CONTROL STATION REFERENCE NAME (ENTER IF NONE)

[]

INPUT LATITUDE IN DDD.MMSSSSS FORMAT (ENTER IF BENCHMARK ONLY)

[] {this station is known elevation and unknown horizontal coordinates}

INPUT CONTROL ELEVATION (Z) (ENTER IF NONE)

[18.23F] {The elevation is known in feet but you have a metric project.
          Simply add a F (survey feet) or I (international feet) and the
          data will be converted for you.  You can do this with horizontal
           plane coordinates, too, and convert meters to either foot units
           if that is required.

ELEVATION CONVERTED TO DESIRED UNITS IS          5.557  {metric equivalent
                                                          of 18.23 ft.}

INPUT CONTROL STATION NAME: (ENTER IF FINISHED)
```

```
[]  {have one more input but it is plane horizontal coordinates so need
     to return to main menu}

1-INPUT BY KEYBOARD /DELETE CONTROL POINTS
2-INPUT/DELETE AZIMUTHS
3-LIST DATA
4-CHANGE (EDIT) EXISTING DATA
5-IMPORT CONTROL FROM .XYZ OR .CTL FILES
6-FREE FORM COORDINATE INPUT
7-CHANGE UNITS (FT., M., INTER. FT.)
8-CHANGE STATE PLANE ZONE
9-IMPORT BY REF. NAME
10-QUIT
CHOOSE A # <10>

[1]  {select input control again}

DO YOU WANT TO?
    1-ADD DATA
    2-DELETE DATA
    3-QUIT
INPUT THE # OF YOUR CHOICE: <3>

[1]

ENTERING A "U" FOR UNDO ALLOWS ONE TO ELIMINATE PREVIOUS ENTRIES


ARE INPUT COORDINATES
(1) LATITUDE , LONGITUDE
(2) NORTHING (Y), EASTING (X)
PICK A # <2>

[]

INPUT CONTROL STATION NAME: (ENTER IF FINISHED)

[H1]

ENTER CONTROL STATION REFERENCE NAME (ENTER IF NONE)

[]

INPUT THE NORTHING (Y) (ENTER IF BENCHMARK ONLY)

[506958.124]

INPUT THE EASTING (X)

[162692.572]
```

INPUT CONTROL ELEVATION (Z) (ENTER IF NONE)

[] {H1 has only known horizontal coordinates}

INPUT CONTROL STATION NAME: (ENTER IF FINISHED)

[] {all finished entering control coordinates}

1-INPUT BY KEYBOARD /DELETE CONTROL POINTS
2-INPUT/DELETE AZIMUTHS
3-LIST DATA
4-CHANGE (EDIT) EXISTING DATA
5-IMPORT CONTROL FROM .XYZ OR .CTL FILES
6-FREE FORM COORDINATE INPUT
7-CHANGE UNITS (FT., M., INTER. FT.)
8-CHANGE STATE PLANE ZONE
9-IMPORT BY REF. NAME
10-QUIT
CHOOSE A # <10>

[2] {Now we will input an azimuth. The azimuth does not have to be at a control station, i.e., it could in the middle of a traverse between two control points.}

DO YOU WANT TO?
1-ADD DATA
2-DELETE DATA
3-QUIT
INPUT THE # OF YOUR CHOICE: <3>

[1]

ENTERING A "U" FOR UNDO ALLOWS ONE TO ELIMINATE PREVIOUS ENTRIES

WILL INPUT BE (1) AZIMUTHS OR (2) BEARINGS - SELECT A # <1>

[1] { Bearing input can be N, S, E, or W for a cardinal direction or quadrant number (1 is NE, 2 is SE, 3 is SW, 4 is NW) followed by a space and the bearing value in dd.mmss format.}

INPUT THE OCCUPIED STATION NAME: (ENTER IF FINISHED)

[H1] {from station of azimuth}

INPUT THE SIGHTED STATION NAME: (ENTER IF FINISHED)

[AZM1] {to station of azimuth}

INPUT AZIMUTH IN DD.MMSSSS FORMAT

```
[273.2356]

OCCUPIED STATION ASSUMED TO BE H1
INPUT SIGHTED STATION NAME: (ENTER IF INCORRECT)

[] {this would allow you to enter more bearings in a traverse sense}

INPUT THE OCCUPIED STATION NAME: (ENTER IF FINISHED)

[] {this would allow you to enter another azimuth from a different station}
   {pressing enter returns you to the main menu}
```

5.4 Listing data

Listing data is performed by list numbers. Each control station or azimuth represents one list number. The data is stored in the order in which it was entered. On a large job you simply want to list only a portion of the data, such as from list # 18 to 32.

```
1-INPUT BY KEYBOARD /DELETE CONTROL POINTS
2-INPUT/DELETE AZIMUTHS
3-LIST DATA
4-CHANGE (EDIT) EXISTING DATA
5-IMPORT CONTROL FROM .XYZ OR .CTL FILES
6-FREE FORM COORDINATE INPUT
7-CHANGE UNITS (FT., M., INTER. FT.)
8-CHANGE STATE PLANE ZONE
9-IMPORT BY REF. NAME
10-QUIT
CHOOSE A # <10>

[3]

DO YOU WANT TO LIST
1-CONTROL STATIONS AND COORDINATES
2-AZIMUTHS
3-QUIT LIST
PICK A # <3>

[1] {you can list control stations or azimuths}

TOTAL NUMBER OF THIS MEASUREMENT TYPE IS     3
INPUT MINIMUM LIST #

[1]

INPUT MAXIMUM LIST #

[3]
```

```
NUMBER OF CONTROL STATIONS IS    3

LIST# STATION         NORTHING (Y)  EASTING (X)     ELEVATION
   REF. NAME           LATITUDE      LONGITUDE
   1 A1                   506082.330   162318.288      9.234
                   28-54- 2.39478  81-23-10.94554
   2 BM1                                               5.557

   3 H1                   506958.124   162692.572
                   28-54-30.88187  81-22-57.23418
PRESS ENTER TO CONTINUE
```

[] {Note 2D control stations show no elevation, and 1D control stations show no 2D coordinates. If a horizontal datum and state plane zone has been selected both state plane and geodetic coordinates will be shown.}

```
1-INPUT BY KEYBOARD /DELETE CONTROL POINTS
2-INPUT/DELETE AZIMUTHS
3-LIST DATA
4-CHANGE (EDIT) EXISTING DATA
5-IMPORT CONTROL FROM .XYZ OR .CTL FILES
6-FREE FORM COORDINATE INPUT
7-CHANGE UNITS (FT., M., INTER. FT.)
8-CHANGE STATE PLANE ZONE
9-IMPORT BY REF. NAME
10-QUIT
CHOOSE A # <10>
```

[3]

```
DO YOU WANT TO LIST
1-CONTROL STATIONS AND COORDINATES
2-AZIMUTHS
3-QUIT LIST
PICK A # <3>
```

[2] {example of azimuth list}

```
TOTAL NUMBER OF THIS MEASUREMENT TYPE IS     1
INPUT MINIMUM LIST #
```

[1]

```
INPUT MAXIMUM LIST #
```

[1]

```
NUMBER OF AZIMUTHS IS   1

LIST#          OCCUPIED            SIGHTED                MEASUREMENT
   1             H1                    AZM1              273-23- 56.0
```

```
PRESS ENTER TO CONTINUE
```

[] {Note bearing input is converted immediately to azimuths and always shown as an azimuth.}

5.5 Changing (editing) data

```
1-INPUT BY KEYBOARD /DELETE CONTROL POINTS
2-INPUT/DELETE AZIMUTHS
3-LIST DATA
4-CHANGE (EDIT) EXISTING DATA
5-IMPORT CONTROL FROM .XYZ OR .CTL FILES
6-FREE FORM COORDINATE INPUT
7-CHANGE UNITS (FT., M., INTER. FT.)
8-CHANGE STATE PLANE ZONE
9-IMPORT BY REF. NAME
10-QUIT
CHOOSE A # <10>
```

[4]

```
(1) CHANGE CONTROL NAME AND COORDINATE INFORMATION
(2) CHANGE AZIMUTH NAME AND MEASUREMENT INFORMATION
(3) CHANGE CONTROL ERROR ESTIMATES
(4) CHANGE AZIMUTH ERROR ESTIMATES
(5) QUIT
SELECT A # <5>
```

[1]

```
DO YOU WANT TO LIST INFORMATION (Y/N) <Y>
```

[N] {List works the same here as in list in the main menu}

```
INPUT LIST # TO EDIT: (ENTER IF FINISHED WITH EDIT OPTION)
```

[2] {BM1 is the second control station.}

```
(1) STATION = BM1      REF. NAME =
(2) NORTHING (Y) =
(3) EASTING (X) =
(4)  LATITUDE =
(5) LONGITUDE =
(6) ELEVATION (Z) =      5.557
(7) QUIT
SELECT A # <7>
```

[6] {You can turn a 1D control point into 3D by adding 2D coordinates. You can remove a portion of a point's coordinates as control by entering zero for its value. For 2-D control you can change plane or geodetic values, and respectively the geodetic or plane values would

```
     automatically change.  No latitude/longitude areas appear if the
    coordinates are assumed.}

INPUT ELEVATION (Z) (ZERO IF NOT VERTICAL CONTROL)
[5.575] {A transposition input blunder is corrected.}

(1) STATION = BM1      REF. NAME =
(2) NORTHING (Y) =
(3) EASTING (X) =
(4)  LATITUDE =
(5) LONGITUDE =
(6) ELEVATION (Z) =      5.575
(7) QUIT
SELECT A # <7>

[] {Note station names can be changed here, or reference names changed or
    added.}

DO YOU WANT TO LIST INFORMATION (Y/N) <Y>

[N]

INPUT LIST # TO EDIT: (ENTER IF FINISHED WITH EDIT OPTION)

[] {No more control coordinates are left to edit.}

(1) CHANGE CONTROL NAME AND COORDINATE INFORMATION
(2) CHANGE AZIMUTH NAME AND MEASUREMENT INFORMATION
(3) CHANGE CONTROL ERROR ESTIMATES
(4) CHANGE AZIMUTH ERROR ESTIMATES
(5) QUIT
SELECT A # <5>

[2] {edit the one input azimuth}

DO YOU WANT TO LIST INFORMATION (Y/N) <Y>

[N]

INPUT LIST # TO EDIT: (ENTER IF FINISHED WITH EDIT OPTION)

[1] {only one azimuth to edit}

(1) OCCUPIED STATION = H1
(2) SIGHTED STATION = AZM1
(3) AZIMUTH = 273-23- 56.00
(4) QUIT
SELECT A # <4>

[2] {Change the sighted station to AZBK5.}
```

INPUT NEW SIGHTED STATION

[AZBK5]

(1) OCCUPIED STATION = H1
(2) SIGHTED STATION = AZBK5
(3) AZIMUTH = 273-23- 56.00
(4) QUIT
SELECT A # <4>

[]

DO YOU WANT TO LIST INFORMATION (Y/N) <Y>

[N]

INPUT LIST # TO EDIT: (ENTER IF FINISHED WITH EDIT OPTION)

[]

5.6 Changing control and azimuth error estimates.

One of the unique characteristics of least squares analysis is that the control can be treated as a measurement with an error estimate, and allowed to adjust. Defaults of 0.001 ft. or m. for control and 0.1 seconds for azimuths are used because these are so small the values are held fixed. There are situations (blunder detection, varying quality, etc.) where one may wish to assign larger error estimates as we all know control is really not perfect. You change error estimates by list number (such as control list number 8 to 12) and you have the ability to list error estimates in this option. When putting in new error values the default value presented to the user is always 0.001 ft./m. or 0.1 seconds (fixed).

5.7 Importing control from other .ctl or .xyz files

Many situations exist where control needs to be imported from another EFBP job. The coordinates would exist either in a control (.ctl) or final (.xyz) coordinate file. You have a choice of importing all stations or simply import by station name. You can import all control as fixed or use the error estimates in the .ctl or .xyz files.

If a geodetic datum and state plane zone has been assigned in both jobs, this import option has the ability to automatically convert the imported coordinates and coordinate error estimates to the correct state plane zone and linear units. Note this option does not transfer between datums (NAD 27 to NAD 83, etc.) but only between state plane zones in the same datum.

As an example the coordinates being imported are in Florida West NAD 83 and are in U.S. survey feet. They are in a file named WEST.CTL and when listed are:

```
NUMBER OF CONTROL STATIONS IS    2

LIST# STATION        NORTHING (Y)  EASTING (X)     ELEVATION
   REF. NAME          LATITUDE      LONGITUDE
   1 WEST1             1666314.963   786888.989
                   28-55- 1.00000  81-35-29.00000
   2 WBM1                                            39.410

{Input occurs in the COE project like:}

1-INPUT BY KEYBOARD /DELETE CONTROL POINTS
2-INPUT/DELETE AZIMUTHS
3-LIST DATA
4-CHANGE (EDIT) EXISTING DATA
5-IMPORT CONTROL FROM .XYZ OR .CTL FILES
6-FREE FORM COORDINATE INPUT
7-CHANGE UNITS (FT., M., INTER. FT.)
8-CHANGE STATE PLANE ZONE
9-IMPORT BY REF. NAME
10-QUIT
CHOOSE A # <10>

[5]

INPUT THE FULL PATH (UNLESS IN EXISTING DIRECTORY)
FOLLOWED BY PROJECT NAME AND EXTENSION (.CTL OR .XYZ)
PRESS ENTER IF FINISHED WITH THIS FUNCTION

[WEST.CTL]  {Note you can access different directories if the entire path
             is used, i.e., C:\EFB\CTL\BREAM.CTL}

IMPORT WITH
(1) EXISTING ERROR ESTIMATES OR
(2) HOLD CONTROL FIXED
SELECT A # <2>

[2]

(1) IMPORT ALL CONTROL OR
(2) IMPORT BY STATION NAME ENTRY
PICK A # <2>

[1]

ADDED    2 CONTROL POINTS AND   0 AZIMUTHS

INPUT THE FULL PATH (UNLESS IN EXISTING DIRECTORY)
FOLLOWED BY PROJECT NAME AND EXTENSION (.CTL OR .XYZ)
PRESS ENTER IF FINISHED WITH THIS FUNCTION
```

[]

{When the coordinates are now listed:}

```
LIST# STATION        NORTHING (Y)  EASTING (X)     ELEVATION
   REF. NAME           LATITUDE      LONGITUDE
   1 A1                  506082.330   162318.288      9.234
                   28-54- 2.39478  81-23-10.94554
   2 BM1                                              5.575

   3 H1                  506958.124   162692.572
                   28-54-30.88187  81-22-57.23418
   4 WEST1               507893.817   239844.244
                   28-55- 1.00000  80-35-29.00000
   5 WBM1                                            12.012
```

Note the latitude and longitude for imported station WEST1 remain the same even though its plane coordinates were converted to Florida East Mercator from Florida West Mercator and also output in meters while in Florida west they were stored in U.S. survey feet. Note also the benchmark WBM1 was converted from feet to meters.

5.8 Import of ascii coordinate files

Since the .ctl is of a unique file structure, a simple ascii control coordinate list cannot be read by EFBP as a control file. Instead CTL allows for a wide variety of ascii file imports when any number of spaces separates data types. The file can be comma delimited except a space is required between station name and the next coordinate in that line. As an example, a common public domain survey processing system called Cadastral Measurement Management (CMM) is often used for control survey processing. It creates two ascii files - a .cor which is station__name X Y and a .lev file which is station_name Z. This example will illustrate import of these two files from a CMM project called CAMPUS into a .ctl which is Maine East Mercator NAD 83. Note the units, datum, and state plane zone in the ascii file and .ctl should match. If they do not, set up the .ctl in the units, datum, and zone of the ascii file then use CTL's utilities to transform it.

```
1-INPUT BY KEYBOARD /DELETE CONTROL POINTS
2-INPUT/DELETE AZIMUTHS
3-LIST DATA
4-CHANGE (EDIT) EXISTING DATA
5-IMPORT CONTROL FROM .XYZ OR .CTL FILES
6-FREE FORM COORDINATE INPUT
7-CHANGE UNITS (FT., M., INTER. FT.)
8-CHANGE STATE PLANE ZONE
9-IMPORT BY REF. NAME
10-QUIT
CHOOSE A # <10>
```

```
[6]

INPUT DATA FORMAT
1 - PT_NAME  X  Y  Z
2 - PT_NAME  N  E  Z
3 - PT_NAME  N  E
4 - PT_NAME  X  Y   (CMM .COR FILE)
5 - PT_NAME  Z  (CMM .LEV FILE)
6 - QUIT
PICK A # <6>

[4]

INPUT FULL PATH AND FILE NAME OF COORDINATE FILE
PRESS ENTER TO QUIT

[C:\CMM\CAMPUS]
{.cor extension is assumed, if different the extension must exist}
 10 CONTROL STATIONS ADDED

1-INPUT BY KEYBOARD /DELETE CONTROL POINTS
2-INPUT/DELETE AZIMUTHS
3-LIST DATA
4-CHANGE (EDIT) EXISTING DATA
5-IMPORT CONTROL FROM .XYZ OR .CTL FILES
6-FREE FORM COORDINATE INPUT
7-CHANGE UNITS (FT., M., INTER. FT.)
8-CHANGE STATE PLANE ZONE
9-IMPORT BY REF. NAME
10-QUIT
CHOOSE A # <10>

[6]

INPUT DATA FORMAT
1 - PT_NAME  X  Y  Z
2 - PT_NAME  N  E  Z
3 - PT_NAME  N  E
4 - PT_NAME  X  Y   (CMM .COR FILE)
5 - PT_NAME  Z  (CMM .LEV FILE)
6 - QUIT
PICK A # <6>

[5]

INPUT FULL PATH AND FILE NAME OF COORDINATE FILE
PRESS ENTER TO QUIT

[C:\CMM\CAMPUS.LEV]
{while .LEV is assumed, it can still be entered - if not .LEV the
extension must be input}
```

```
STATION BOOK     ALREADY EXISTS ADDED IF MISSING SOME COOR.
STATION CENTER   ALREADY EXISTS ADDED IF MISSING SOME COOR.
STATION CORNER   ALREADY EXISTS ADDED IF MISSING SOME COOR.
STATION CROW     ALREADY EXISTS ADDED IF MISSING SOME COOR.
STATION DORM     ALREADY EXISTS ADDED IF MISSING SOME COOR.
STATION KEY      ALREADY EXISTS ADDED IF MISSING SOME COOR.
STATION KNOTT    ALREADY EXISTS ADDED IF MISSING SOME COOR.
STATION POST     ALREADY EXISTS ADDED IF MISSING SOME COOR.
STATION RAY      ALREADY EXISTS ADDED IF MISSING SOME COOR.
STATION RICH     ALREADY EXISTS ADDED IF MISSING SOME COOR.
  0 CONTROL STATIONS ADDED
{The following messages occurred because those stations existed from the
.cor import, but they had no elevations till the .lev was imported.}

1-INPUT BY KEYBOARD /DELETE CONTROL POINTS
2-INPUT/DELETE AZIMUTHS
3-LIST DATA
4-CHANGE (EDIT) EXISTING DATA
5-IMPORT CONTROL FROM .XYZ OR .CTL FILES
6-FREE FORM COORDINATE INPUT
7-CHANGE UNITS (FT., M., INTER. FT.)
8-CHANGE STATE PLANE ZONE
9-IMPORT BY REF. NAME
10-QUIT
CHOOSE A # <10>

[1]

DO YOU WANT TO LIST
1-CONTROL STATIONS AND COORDINATES
2-AZIMUTHS
3-QUIT LIST
PICK A # <3>

[1]

TOTAL NUMBER OF THIS MEASUREMENT TYPE IS    10

INPUT MINIMUM LIST #

[1]

INPUT MAXIMUM LIST #

[10]

NUMBER OF CONTROL STATIONS IS   11

LIST# STATION        NORTHING (Y)  EASTING (X)     ELEVATION
   REF. NAME          LATITUDE      LONGITUDE
```

```
   1 BOOK              449673.509   940816.383    112.809
                  44-54- .50335  68-40- 3.46269
   2 CENTER            450684.005   941087.973    115.499
                  44-54-10.48734  68-39-59.71808
   3 CORNER            451466.452   942390.324    145.777
                  44-54-18.23986  68-39-41.64417
   4 CROW              451342.069   942412.564    145.762
                  44-54-17.01203  68-39-41.33171
   5 DORM              450836.237   942397.549    150.307
                  44-54-12.01674  68-39-41.52635
   6 KEY               450593.009   940717.277    117.392
                  44-54- 9.58122  68-40- 4.86610
   7 KNOTT             451290.032   940968.780    115.633
                  44-54-16.46932  68-40- 1.39154
   8 POST              451439.688   941919.320    147.414
                  44-54-17.96626  68-39-48.18804
   9 RAY               449828.874   942256.130    125.620
                  44-54- 2.06643  68-39-43.46337
  10 RICH              449847.018   941249.300    114.475
                  44-54- 2.22551  68-39-57.45274
PRESS ENTER TO CONTINUE
```

5.9 Change units

This option is #7 in the main menu. It simply changes the entire control data set from meters to survey feet, survey feet to meters, meters to international feet, international feet to meters, survey feet to international feet, or international feet to survey feet.

5.10 Change state plane zone

This option is #8 in the main menu. It simply changes the entire control data set from one state plane zone to another. The units of the original data are retained. The unit and zone change combine to let you do things like convert a control file from Florida North Lambert NAD 83 in survey feet to Florida East Mercator NAD 83 meters. Note no transformation between NAD 27 and NAD 83 is provided in CTL.

5.11 Use of reference names

The concept of a reference name is that the field survey name in EFB is a temporary name assigned in the field. The reference name could be the actual control station name on its brass cap, or some other form of easier identification due to a longer and more descriptive name. It would allow two field crews working on the same job to not have to worry about overlapping survey point names, as instead the reference name would indicate which control stations were commonly observed. A surveyor using EFB can assign reference names in the field (.obs) , and a surveyor using CTL can assign reference names to entered control (.ctl).

Reference names can be used in three ways.

The first is by use of reference names in .obs only. CTL finds those stations in .obs and places them in control with arbitrary control coordinates. A surveyor can then use CTL to replace the arbitrary coordinates with actual control values which are derived from the office person's recognition of the reference name.

The second option is reading in a .ctl or .xyz file into the existing .ctl. This will only import stations with reference names and the field survey names in those file(s) will be retained.

The third option is matching reference names in a .ctl or .xyz to the project's .obs reference names. The field survey names in the .ctl or .xyz will be replaced with those in .obs . This procedure requires an exact match (spelling) of the reference name in .obs to the selected .ctl . This option could be used if a control traverse was processed with EFBP and all reference names were filled in the .xyz . Topographic surveying from this control could now occur and reference names could be used in .obs so that CTL will be able to select the correct control coordinates from the control traverse .xyz file.

The input of the appropriate file names for each of the options is similar to file name input previously and thus requires no examples.

5.12 Format of the control (.ctl) file

The .ctl for project COE after the manual input of three control stations and one azimuth, editing of a benchmark elevation, and import of two more control stations from another .ctl follows. The numbered line is for reference to column specific items (names left justified, decimal point column important for coordinates and error estimates).

```
123456789112345678921234567893123456789412345678951234567896123456789712
G 00 COE
G 01 A1                                                            0901 83M29
G 02   162318.288   506082.330      9.234     .001     .001     .001
G 01 BM1                                                           0901 83M29
G 02                                5.575                           .001
G 01 H1                                                            0901 83M29
G 02   162692.572   506958.124               .001     .001
G 03 AZBK5                    273 23 56.0     .10  G
G 01 WEST1                                                         0901 83M29
G 02   239844.244   507893.817               .001     .001
G 01 WBM1                                                          0901 83M29
G 02                               12.012                           .001
```

The first line (G 00) is a header record which contains the project name. Each station consists of two lines with an optional lines for azimuths. The G 01 line contains station name, reference name (optional, begins in column 15), state plane NGS zone number, horizontal datum, units (M, F, or

I), and vertical datum. The geodetic items are optional if assumed coordinates are being used.

The G 02 record contains X (easting), Y (northing), Z (elevation), X error estimate, Y error estimate, and Z error estimate. If a coordinate area is blank that means it is unknown. If a coordinate error estimate area is blank the error estimate defaults to fixed (0.001).

The azimuth record ("to" station) is a G 03 record and follows the G 02 record for the from station of the azimuth. It contains to station name, azimuth (ddd mm ss.s), error estimate (decimal seconds), and type (G - grid or A -astronomic). If the error estimate is blank 0.10 seconds (fixed) is assumed.

Chapter 6. Discussion of EFBP menu options

6.1 Introduction

EFBP processing options are selected immediately after typing EFBP at the DOS prompt and pressing enter. These options are stored in two files in the directory you are in. These files are named DEFAULT.CON and DEFAULT.SD. If these files do not exist EFBP creates them with assignment of "reasonable" values.

6.2 Example

This is the example menu for option selections in EFBP.

```
(1) PROJECT NAME IS BRIDGE
(2) USE REPETITION ERRORS PLUS ADD-ONS IN ERROR ESTIMATION
(3) DO NOT COMPUTE COORDINATE STANDARD ERRORS AND ERROR ELLIPSES
(4) CORRECT FOR EARTH CURVATURE AND ATMOSPHERIC REFRACTION
(5) ROBUST ERROR ESTIMATE PROMPT WILL NOT APPEAR
(6) PROCESS TO FINAL COORDINATE .XYZ FILE

FOLLOWING USED AS ADD-ONS TO ERROR FROM REPETITION
DISTANCE   DISTANCE     HORZ.     AZIMUTH   TRIG. LEV.   TRIG. LEV.  DIFF.
LEV.
CONSTANT      PPM      ANGLE (SEC)   (SEC)      CONSTANT         PPM
CONSTANT
(7)  .004 (8)  5.00 (9)   4.0   (10)    .1 (11)  .008   (12) 20.00  (13)
.010

FOLLOWING ARE USER DEFINED ERROR ESTIMATES
  DISTANCE   DISTANCE     HORZ.      AZIMUTH    TRIG. LEV.   DIFF. LEV.
  CONSTANT     PPM     ANGLE (SEC)    (SEC)      CONSTANT     CONSTANT
(14)  .006 (15)  5.00   (16)  10.0 (17)    .1   (18)  .020   (19)  .010

(20) SETUP ERROR (ALWAYS USED) =   .005
(21) READ ERROR ESTIMATE ADD-ONS FROM DIFFERENT DEFAULT.CON
(22) READ USER DEFINED ERROR ESTIMATES FROM DIFFERENT DEFAULT.SD
FLAG MAXIMUM SPREADS ABOVE
(23) DISTANCE =   .005 (24) ANGLES =   10.0 (25) ELEV. DIFFERENCES =   .020
ENTER A # TO CHANGE, OR PRESS ENTER TO START PROCESSING
```

6.3 Explanation of example

Explanations for each option follows. Note selection of datum and units (ft. vs. m.) is performed via control using program CTL which modifies the project .ctl (control) file.

(1) PROJECT NAME IS BRIDGE

You can change the project name. Note the .obs (measurement file) is required to exist in the working directory. A .ctl (control file) is also normally required though limited forms of processing can be performed without control.

(2) USE REPETITION ERRORS PLUS ADD-ONS IN ERROR ESTIMATION
You can use repetition errors plus user defined add-ons for error estimation in the least squares analysis. If you shut this option off you will be using user defined error estimates without any influence from repetition error.

With this option on error estimates of control coordinates will be read from the control file .ctl. If this option is turned off control coordinates will be held fixed (error estimate of 0.001 ft./m.) independent of error estimates in .ctl.

(3) DO NOT COMPUTE COORDINATE STANDARD ERRORS AND ERROR ELLIPSES
Least squares enables computation of coordinate standard errors and error ellipses. While this information can be of use, for many applications they are not required. If this option is off (do not compute) the least squares solution will operate faster as less computations are necessary.

(4) CORRECT FOR EARTH CURVATURE AND ATMOSPHERIC REFRACTION
If your raw survey measurements are not corrected for these systematic errors you will normally desire that the correction be applied. If for some reason you note the zenith angles and slope distances in your .obs file are already corrected you will want this option off.

(5) ROBUST ERROR ESTIMATE PROMPT WILL NOT APPEAR
Robustness is a re-weighting technique used in least squares to try to identify blunders. It is simply strategy which uses the residuals (amounts of adjustment) in making weaker data have less effect on the adjustment by giving the weaker data larger error estimates. You will only want this option on when you want to take advantage of this option.

(6) PROCESS TO FINAL COORDINATE .XYZ FILE
You have 4 levels of processing and you can quit terminating at any selected stage:
1st stage - abstracting of repeated measurements and preliminary coordinate generation - this produces the .GEN report
2nd stage - 1D least squares - this produces the .1D report
3rd stage - 2D least squares - this produces the .2D report
4th stage - final coordinate and attribute production - .XYZ file

FOLLOWING USED AS ADD-ONS TO ERROR FROM REPETITION

DISTANCE CONSTANT	DISTANCE PPM	HORZ. ANGLE (SEC)	AZIMUTH (SEC)	TRIG. LEV. CONSTANT	TRIG. LEV. PPM	DIFF. LEV. CONSTANT
(7) .004	(8) 5.00	(9) 4.0	(10) .1	(11) .008	(12) 20.00	(13) .010

If use of repetition errors is selected, these values are the add-ons to the errors from repetition for least squares error estimation. For example a horizontal distance repetition error of .005 m. will have .004 m. added to it. Note since trig. elevation differences usually erode in accuracy quicker as a function of distance when compared to the horizontal distance, add-on ppm's (parts per million) are separated into horizontal distance and trig. leveling where the trig. leveling value is usually larger in magnitude.

FOLLOWING ARE USER DEFINED ERROR ESTIMATES

DISTANCE CONSTANT	DISTANCE PPM	HORZ. ANGLE (SEC)	AZIMUTH (SEC)	TRIG. LEV. CONSTANT	DIFF. LEV. CONSTANT
(14) .006	(15) 5.00	(16) 10.0	(17) .1	(18) .020	(19) .010

If error estimation from repetition is not selected, or if a measurement is not repeated and thus no repetition error exists, the user defined error estimates will be utilized.

(20) SETUP ERROR (ALWAYS USED) = .005

It is well known that on shorter lines it is more difficult to point reliably in an angular sense than on long lines. As an example a 2 second least count total station does not have 2 second pointing reliability on a 10 m. line. Setup error is added to all horizontal angle and azimuth measurement error estimates (both from repetition and user defined) and is computed by the tangent inverse of the setup error divided by the length of the line. A zero setup error indicates you do not want this type of value added to your error estimates.

(21) READ ERROR ESTIMATE ADD-ONS FROM DIFFERENT DEFAULT.CON

DEFAULT.CON is where error estimate add-ons are read from. You may want values from an old job read into your present job, or you may be reading some metric default values instead of the English values from the last job you processed. The DEFAULT.CON you are reading from can be in a different path/directory and have a different file name.

(22) READ USER DEFINED ERROR ESTIMATES FROM DIFFERENT DEFAULT.SD
DEFAULT.SD is where user defined error estimates are read from. You may want values from an old job read into your present job, or you may be reading some metric default values instead of the English values from the last job you processed. The DEFAULT.SD you are reading from can be in a different path/directory and have a different file name.

FLAG MAXIMUM SPREADS ABOVE
(23) DISTANCE = .005 (24) ANGLES = 10.0 (25) ELEV. DIFFERENCES = .020

The .GEN report can become very large as all repetition errors are reported.

Since you really only want to study the "poor" values, you can flag with asterisks (* *) maximum spreads above defined tolerances. When reviewing the report in a text editor you can put the editor in search mode for asterisks. A maximum spread is the largest difference of a set of repetitions from the average value.

6.4 Expanded discussion of some EFBP options

6.4.1 Earth curvature / atmospheric refraction correction

Regarding earth curvature and atmospheric refraction, it is a squared function of the length of the line - a 1000 ft. shot has 4 times (.02 ft) the correction of a 250 ft. shot. (.005 ft.). Thus it is usually smaller than the random error in our trig. leveling and this is why not correcting does not create a significant error. Another reason not to correct it is if someone computes the elevation difference on their calculator without curvature and refraction correction it will not be equal to the value derived from processing with curvature and refraction turned on.
The correction is always positive because: assume a H.I. of zero and a zenith angle of 90 the further out you go the larger the H.T. will be to keep that "flat" line of sight due to earth curvature. That H.T. which is subtracted must be negated with a positive correction to produce an elevation difference of zero. Note it is possible with the correction for residuals to get a little worse - this simply shows the random error is bigger than the correction.

6.4.2 Robustness

Robustness is a re-weighting strategy which can be used to find blunders as "the weak get weaker and the strong get stronger." Assume when the 2-D least squares runs two angles have error estimates of 5 seconds and 30 seconds respectively, and the residuals are 45 seconds and 3 seconds respectively. When you robust the new error estimate of the first angle will be (5+45)/2=25 seconds and the second angle will be (30+3)/2=16.5 seconds. The adjustment re-runs with the new error estimates.

It is recommended to use robustness only as a tool in blunder detection, not for the production of final coordinates. It will not usually find real huge blunders as caused by station naming.

If you have robustness turned on at the end of the least squares portion of LEVEL and LSAQ you will asked if you want to re-weight. Answering Y (yes) to this performs the robustness on all measurements and the least squares re-runs. All measurements are robusted (control, elev. diff., distances, angles, azimuths) though in an update EFBP could allow you to do things like robust distances only. You can robust as many times as you want. Robustness aids in finding the problem if the standard error of unit weight jumps way down.

The .XYZ will reflect the coordinates from the last robustness but the standard deviations and error ellipses are from the first least squares.
You should re-process without robusting for your last run anyway.

6.4.3 Different ppm in error estimation for horizontal distances and trig.

elevation differences

In previous versions of EFBP the ppm was the same for horizontal distances and trig. leveling derived from HVD data. We all know longer lines get worse faster in elevation differences than horizontal distances. Assume trig. leveling and horizontal distance ppm's are 30 and 1 respectively. Thus the 30 vs. 1 means a ppm add-on for a 300 ft. and a 3000 ft. line are horizontally (0.0003 ft. vs. 0.003 ft.) and vertically (.009 ft. vs. 0.09 ft.). This will allow you to shoot HVD mode (not HD) on long line as the larger error estimate on the long line elevation differences will make it have less affect on your final coordinates. This is the power of effective error estimation.

6.4.4 Setup error

Note setup error is in ft. or m. and is an add-on to all horizontal angles and azimuths. Short lines get a bigger effect - long lines will get essentially no effect. This is again very effective as you do not want small error estimates on 50 ft. backsights or foresights. A .005 ft. setup error amounts to 21 sec. of error in 50 ft., 2 sec. in 500 ft., and 0.2 sec. in 5000 ft. (tangent inverse function). The setup error is always applied, so if you do not like it, make it zero. One way to think of setup error is in terms of your ability to set up over a point with your tripod, tribrach, and optical plummet.

Chapter 7. Introduction to EFB processing.

EFBP can be described as a four step process. You can instruct the program to stop at any step, and the program moves between steps with no required input from the user. Obviously one needs to set up the appropriate parameters in the EFBP setup menu before processing can begin.

The first step of EFBP is abstracting, horizontal sideshot identification, and preliminary horizontal traverse closure and coordinate generation. This process produces the first report file .gen . Abstracting involves:

(1) averaging of multiple repetitions and computing of respective error statistics,

(2) reducing raw data to 2-D and 1-D measurement equivalents,

(3) computing initial error estimates for all values, and

(4) comparing horizontal distances and elevations on lines measured multiple times and weighted averaging their values based on error estimates. Weighted averaging permits the size of the error estimate to be used in the averaging process so better indicators of quality measurements have more affect in the averaging process.

The redundant part of the horizontal data is then identified (non-sideshots). The program then automatically performs traverse closures until every redundant horizontal station has coordinates. These coordinates serve as the required estimates for the 2-D least squares adjustment to operate.

The second part of processing is generation of all elevation information. This procedure creates the .1d report file. First the redundant portion of the vertical component of the survey network is identified, and a least squares analysis of this data is performed. Sideshot elevation determinations are then computed based on the least squares adjusted elevations.

The third part of processing is the 2-D least squares. This produces the .2d report file. All reductions to state plane are performed automatically if a zone and datum are defined. Elevation reduction to grid is possible because these values have been generated in the previous step. Traverse closures are then computed which compare raw survey measurements (reduced to grid if that is defined) to adjusted coordinates.

The fourth part of processing is two distinct operations. The first part computes the horizontal sideshots based on the least squares adjusted coordinates of the redundant survey. Grid reductions are again automatically performed if required. The second portion merges the coordinate information with the attribute information in generation of the final .xyz file. If SOR, SO, (station offset) or R (differential leveling) data exists a .soe (station, offset, elevation) report file is generated.

Chapter 8. Examination of processing's first report file - .GEN

8.1 Example

A 1726 station topographic route job is used as an example for the next four chapters. Note this is a metric job so all numerical values which reference coordinates, elevations, elevation differences, distances, etc. are in meters. The definition of metric units made in the control (.ctl) file, is reported in the horizontal least squares (.2d) report, and final coordinates are labelled as metric in the final coordinate (.xyz) and station-offset-elevation (.soe) files. The first report (.gen) details abstracting and preliminary coordinate generation. For a job this large the .gen report is lengthy so only key items are shown here as they represent all the types of information generally shown in this report.

```
PROJECT EXAMP    PARAMETERS
USE REPETITION ERRORS PLUS ADD-ONS IN ERROR ESTIMATION
COMPUTE COORDINATE STANDARD ERRORS AND ERROR ELLIPSES
CORRECT FOR EARTH CURVATURE AND ATMOSPHERIC REFRACTION
ROBUST ERROR ESTIMATE PROMPT WILL NOT APPEAR
PROCESS TO FINAL COORDINATE .XYZ FILE

FOLLOWING USED AS ADD-ONS TO ERROR FROM REPETITION
DISTANCE   DISTANCE     HORZ.     AZIMUTH   TRIG. LEV.   TRIG. LEV.  DIFF. LEV
CONSTANT     PPM     ANGLE (SEC)   (SEC)     CONSTANT        PPM      CONSTANT
     .005      5.00       6.0         10.0       .010        50.00        .010

FOLLOWING ARE USER DEFINED ERROR ESTIMATES
  DISTANCE   DISTANCE     HORZ.      AZIMUTH    TRIG. LEV.   DIFF. LEV.
  CONSTANT     PPM     ANGLE (SEC)    (SEC)      CONSTANT     CONSTANT
      .007       5.00         12.0       10.0         .030         .010

SETUP ERROR (ALWAYS USED) =   .002
FLAG MAXIMUM SPREADS ABOVE
(23) DISTANCE =   .020 (24) ANGLES =   20.0 (25) ELEV. DIFFERENCES =   .030


HORZ. COLLIMATION CORRECTION =     -4.7 SECONDS
VERT. COLLIMATION CORRECTION =     -3.8 SECONDS
HORIZONTAL POINTING STANDARD ERROR  (DIRECT) =       5.0  SECONDS
  VERTICAL POINTING STANDARD ERROR  (DIRECT) =       2.1  SECONDS
HORIZONTAL POINTING STANDARD ERROR (REVERSE) =       5.0  SECONDS
  VERTICAL POINTING STANDARD ERROR (REVERSE) =       3.8  SECONDS


PROCESSING SETUP #   1 AT STATION A1
REPETITION ERROR ON MULTIPLE POINTING TO STATION A2       IS       1.0  SEC.
REPETITION ERROR ON MULTIPLE POINTING TO STATION A2       IS        .0  SEC.
REPETITION STANDARD ERRORS
SIGHTED      HORIZONTAL DISTANCE      ELEVATION DIFF.           COMPARE
STATION      SD     SD     MAX        SD    SD      MAX      HORZ.     ELEV.
                (MEAN)  SPREAD          (MEAN)   SPREAD     DIST.     DIFF.
A2         .001    .000    .001     .008   .004     .010
REPETITION STANDARD ERRORS FOR ANGLES
BS STATION     FS STATION     SD    SD (MEAN)  MAX SPREAD
WARNING - CALIBRATION RECORD WITHOUT DATA

PROCESSING SETUP #   2 AT STATION A1
REPETITION ERROR ON MULTIPLE POINTING TO STATION A2       IS        .0  SEC.
REPETITION ERROR ON MULTIPLE POINTING TO STATION A2       IS       1.0  SEC.
REPETITION STANDARD ERRORS
SIGHTED      HORIZONTAL DISTANCE      ELEVATION DIFF.           COMPARE
STATION      SD     SD     MAX        SD    SD      MAX      HORZ.     ELEV.
                (MEAN)  SPREAD          (MEAN)   SPREAD     DIST.     DIFF.
A2         .001    .000    .001     .005   .002     .007     -.004      .005
REPETITION STANDARD ERRORS FOR ANGLES
BS STATION     FS STATION     SD    SD (MEAN)  MAX SPREAD

PROCESSING SETUP #   3 AT STATION A1
REPETITION ERROR ON MULTIPLE POINTING TO STATION A2       IS       4.0  SEC.
REPETITION ERROR ON MULTIPLE POINTING TO STATION A2       IS       8.0  SEC.
REPETITION STANDARD ERRORS
SIGHTED      HORIZONTAL DISTANCE      ELEVATION DIFF.           COMPARE
STATION      SD     SD     MAX        SD    SD      MAX      HORZ.     ELEV.
                (MEAN)  SPREAD          (MEAN)   SPREAD     DIST.     DIFF.
```

```
A2         .000    .000    .001     .005   .002     .007     -.002      .012
REPETITION STANDARD ERRORS FOR ANGLES
BS STATION     FS STATION     SD    SD (MEAN)  MAX SPREAD

PROCESSING SETUP #   1 AT STATION A2
REPETITION ERROR ON MULTIPLE POINTING TO STATION A1       IS       4.0  SEC.
REPETITION ERROR ON MULTIPLE POINTING TO STATION A1       IS       1.0  SEC.
REPETITION ERROR ON MULTIPLE POINTING TO STATION BM1      IS       7.0  SEC.
REPETITION ERROR ON MULTIPLE POINTING TO STATION BM1      IS       4.0  SEC.
REPETITION STANDARD ERRORS
SIGHTED      HORIZONTAL DISTANCE      ELEVATION DIFF.           COMPARE
STATION      SD     SD     MAX        SD    SD      MAX      HORZ.     ELEV.
                 (MEAN)  SPREAD          (MEAN)   SPREAD     DIST.     DIFF.
A1         .001    .001    .002     .005   .003     .008      .000  *  -.042*
BM1        .000    .000    .000     .001   .000     .001
REPETITION STANDARD ERRORS FOR ANGLES
BS STATION     FS STATION     SD    SD (MEAN)  MAX SPREAD
A1             BM1             2.        1.          1.


 1714 OF  1726 STATIONS ARE HORIZONTAL SIDESHOTS

TRAVERSE CLOSURE REPORT
LINEAR ERROR OF CLOSURE IS         .048 FT.

PRECISION IS 1/       17355.


STATION                  X COOR.         Y COOR.

A2                    495149.598      128318.648
TRAV1                 495281.157      128298.531
A3                    495980.147      128287.811

TRAVERSE CLOSURE REPORT
LINEAR ERROR OF CLOSURE IS         .051 FT.

PRECISION IS 1/       16398.


STATION                  X COOR.         Y COOR.

A2                    495149.598      128318.648
TRAV2                 495631.118      128293.637
A3                    495980.147      128287.811


STATION  BM1      HAS COORDINATES GENERATED BY ANGLE AND DISTANCE
FROM KNOWN BACKSIGHT
X=   495201.372 Y=   128318.744


STATION  BM7      HAS COORDINATES GENERATED BY ANGLE AND DISTANCE
FROM KNOWN BACKSIGHT
X=   496993.652 Y=   128259.200


FINAL GENERATED COORDINATES

STATION                   X COOR.         Y COOR.

A1                     494886.316      128284.789
A2                     495149.598      128318.648
A3                     495980.147      128287.811
A4                     496811.525      128276.865
A5                     497318.565      128280.773
BM1                    495201.372      128318.744
TRAV1                  495281.157      128298.531
TRAV2                  495631.118      128293.637
TRAV3                  496404.186      128282.819
BM7                    496993.652      128259.200
TRAV4                  497080.913      128272.918
TRAV5                  497330.332      128286.047

DISTANCE RESIDUALS

            DISTANCE                   RESIDUAL

            A1       - A2                  .016
            A2       - BM1                 .000
            A2       - TRAV1               .008
            TRAV1    - A3                  .040
            TRAV1    - BM1                 .004
            A3       - A2                  .046
            A3       - TRAV2               .021
            TRAV2    - A2                  .029
            A3       - A4                  .042
```

```
A3       - TRAV3               .024
TRAV3    - A4                  .023
A4       - A5                  .030
A4       - BM7                 .000
A4       - TRAV4               .016
TRAV4    - A5                  .014
TRAV4    - BM7                 .013
TRAV4    - TRAV5               .017
TRAV5    - A4                  .029
TRAV5    - A5                  .001
```

```
ANGLE RESIDUALS

     ANGLE                              RESIDUAL (SEC)

     A1       - A2       - BM1              .0
     A1       - A2       - TRAV1          -3.1
     A2       - TRAV1    - A3               .0
     A2       - TRAV1    - BM1            -4.7
     A2       - A3       - TRAV2          -1.8
     A2       - TRAV2    - A3               .0
     A4       - A3       - TRAV2           2.0
     A4       - A3       - TRAV3           2.1
     A3       - TRAV3    - A4               .0
     TRAV3    - A4       - A5             -1.4
     TRAV3    - A4       - BM7              .0
     TRAV3    - A4       - TRAV4          -8.9
     A4       - TRAV4    - A5               .0
     A4       - TRAV4    - BM7           -15.0
     A4       - TRAV4    - TRAV5           5.4
     A4       - TRAV5    - A5               .0
```

Individual components of the .GEN report are now detailed.

8.2 EFBP setup parameters

```
PROJECT EXAMP    PARAMETERS
USE REPETITION ERRORS PLUS ADD-ONS IN ERROR ESTIMATION
COMPUTE COORDINATE STANDARD ERRORS AND ERROR ELLIPSES
CORRECT FOR EARTH CURVATURE AND ATMOSPHERIC REFRACTION
ROBUST ERROR ESTIMATE PROMPT WILL NOT APPEAR
PROCESS TO FINAL COORDINATE .XYZ FILE

FOLLOWING USED AS ADD-ONS TO ERROR FROM REPETITION
DISTANCE   DISTANCE     HORZ.     AZIMUTH   TRIG. LEV.   TRIG. LEV.  DIFF. LEV
CONSTANT     PPM     ANGLE (SEC)   (SEC)     CONSTANT        PPM      CONSTANT
     .005      5.00       6.0         10.0       .010        50.00        .010

FOLLOWING ARE USER DEFINED ERROR ESTIMATES
  DISTANCE   DISTANCE     HORZ.      AZIMUTH    TRIG. LEV.   DIFF. LEV.
  CONSTANT     PPM     ANGLE (SEC)    (SEC)      CONSTANT     CONSTANT
      .007      5.00         12.0       10.0          .030         .010

SETUP ERROR (ALWAYS USED) =   .002
FLAG MAXIMUM SPREADS ABOVE
(23) DISTANCE =   .020 (24) ANGLES =   20.0 (25) ELEV. DIFFERENCES =   .030
```

This first section always exists at the start of a .gen file. It echoes the default parameters which were established in the initial menu of EFBP. This information is critical in the exact reproducibility of final coordinates.

8.3 Instrument calibration

```
HORZ. COLLIMATION CORRECTION =     -4.7 SECONDS
VERT. COLLIMATION CORRECTION =     -3.8 SECONDS
HORIZONTAL POINTING STANDARD ERROR  (DIRECT) =       5.0  SECONDS
  VERTICAL POINTING STANDARD ERROR  (DIRECT) =       2.1  SECONDS
HORIZONTAL POINTING STANDARD ERROR (REVERSE) =       5.0  SECONDS
  VERTICAL POINTING STANDARD ERROR (REVERSE) =       3.8  SECONDS
```

This is output from a calibration record which contained a numerical calibration of a total station which involves pointing at a discrete point at least once in the direct and reverse position. Multiple pointings ensures the quality of the calibration and provides an estimate of the observer's pointing ability. The pointing errors are the standard deviations of multiple pointings and would be tagged with asterisks if above user defined tolerances in the EFBP default settings.

If the instrument was in perfect calibration and an operator was perfect in pointing, the sum of the zenith circle readings in direct and reverse would be 360 degrees. Similarly, the direct and reverse horizontal circle readings would differ by exactly 180 degrees. The corrections are one-half the difference between these ideal calibration values.

EFBP reduces individual horizontal circle readings to horizontal angles uniquely for each position number and face (direct or reverse) before averaging. This process cancels the effects of horizontal collimation error. The calibration corrections in the vertical direction are much more critical due to our inability to level trigonometrically as precisely as we measure horizontally.

8.4 Multiple pointings, standard deviations, maximum spreads, and compare multiple lines

```
PROCESSING SETUP #   1 AT STATION A2
REPETITION ERROR ON MULTIPLE POINTING TO STATION A1       IS       4.0  SEC.
REPETITION ERROR ON MULTIPLE POINTING TO STATION A1       IS       1.0  SEC.
REPETITION ERROR ON MULTIPLE POINTING TO STATION BM1      IS       7.0  SEC.
REPETITION ERROR ON MULTIPLE POINTING TO STATION BM1      IS       4.0  SEC.
REPETITION STANDARD ERRORS
SIGHTED      HORIZONTAL DISTANCE      ELEVATION DIFF.          COMPARE
STATION      SD      SD      MAX        SD     SD      MAX      HORZ.     ELEV.
                   (MEAN)  SPREAD          (MEAN)   SPREAD     DIST.     DIFF.
A1         .001    .001    .002     .005   .003     .008      .000  *  -.042*
BM1        .000    .000    .000     .001   .000     .001
REPETITION STANDARD ERRORS FOR ANGLES
BS STATION     FS STATION     SD    SD (MEAN)  MAX SPREAD
A1             BM1            2.       1.          1.
```

A setup and its observations can create information such as this. The setup is always described by its point name and the setup number for that station name. The setup number is very important in reviewing the .gen and matching that information to what is in .obs . As an example a problem in the third setup at station A15 allows one to move in the .obs file past the first two setups at A15 to reach the setup of concern.

Immediately after the setup record is multiple pointing information, which is the difference in horizontal circle reading between two pointings with the same position number with the same telescope orientation. In the preceeding example this setup was occupied for a long time while collecting a significant amount of topographic detail. As a check the surveyor routinely returned to the backsight(s) to assure that the horizontal circle or instrument setup has not been disturbed. In this case the surveyor was using A1 as a horizontal backsight (a long distance) and BM1 for a vertical backsight (a short distance). Multiple pointing spreads would be asterisked if above user defined tolerances. Note this multiple pointing error is with respect to horizontal circle readings, not horizontal angles.

Next in the setup report are repetition errors resulting from multiple observations where the values are reduced to horizontal distances, elevation differences (mark to mark), and horizontal angles. EFBP selects the backsight station as the point which is observed the most times at that setup. If different points are observed the same number of times, the first of those points in the observation list for that setup is selected as the backsight as that generally is the first station sighted in chronological order.

Repetition errors are simply derived in a simple averaging process. Due to the ability to correct for instrument systematic errors based on calibration, direct and reverse readings are treated as individual observations since they are corrected for instrument systematic errors. Older procedures for averaging used to average common direct and reverse readings, and then average those values, because instrument calibration values were not automatically being corrected. While standard errors are important, in blunder identification the maximum (max) spread is of more important concern as it details the largest deviation from the average of any single measurement. This value would be asterisked if above user defined tolerance level.

Of special importance are the compare horizontal distance and elevation information. This area compares the present setup's averaged values to a previous setup which contains the occupied and sighted station. This normally occurs when a prism is used on a backsight as distance and elevation change are thus measured in both directions on a line, and is an incredibly reliable check on instrument setup, height of instrument, and height of target. The comparison is also made if a setup is occupied multiple times and backsight(s) contain prism(s). In this example A1 had been previously occupied and A2 was measured to in HVD mode so this type of comparison can be made. The elevation difference comparison is asterisked because 0.030 m. was the tolerance for this comparison. This is an example of where survey judgement is needed to not be alarmed by the asterisks since a very long line makes it difficult to measure elevation change precisely. At that setup the shorter line to the benchmark will be the more controlling observation for elevation determination. If a line has been observed more than twice, the comparison is made to the first determination on the line for elevation differences, and the horizontal distance comparison is made to the existing weighted average distance for that line.

8.5 Number of horizontal sideshots

```
 1714 OF  1726 STATIONS ARE HORIZONTAL SIDESHOTS
```

This message is an important indicator as the surveyor is generally able to estimate how many traverse stations exist. If this indicator appears incorrect, a station naming problem usually exists as it often causes stations to be "connected" to stations in .obs which cannot be possible based on your knowledge of the field survey. A horizontal sideshot station is defined as a station without horizontal control coordinates that is attached to only one other station by one angle and one distance measurement. A spur traverse (series of setups ending in a sideshot) is recognized as all sideshots because the algorithm "peels" sideshots in an iterative fashion until no sideshots remain.

8.6 Traverse closure report

```
TRAVERSE CLOSURE REPORT
LINEAR ERROR OF CLOSURE IS        .048 M.

PRECISION IS 1/      17355.


STATION                  X COOR.        Y COOR.

A2                    495149.598     128318.648
TRAV1                 495281.157     128298.531
A3                    495980.147     128287.811

TRAVERSE CLOSURE REPORT
LINEAR ERROR OF CLOSURE IS        .051 M.

PRECISION IS 1/      16398.


STATION                  X COOR.        Y COOR.

A2                    495149.598     128318.648
TRAV2                 495631.118     128293.637
A3                    495980.147     128287.811


STATION  BM1      HAS COORDINATES GENERATED BY ANGLE AND DISTANCE
FROM KNOWN BACKSIGHT
X=   495201.372 Y=   128318.744


STATION  BM7      HAS COORDINATES GENERATED BY ANGLE AND DISTANCE
FROM KNOWN BACKSIGHT
X=   496993.652 Y=   128259.200
```

Once the sideshots have been removed, EFBP is required to identify how to generate coordinates on all remaining points. This is necessary as the horizontal least squares adjustment needs preliminary (approximate) coordinates for all unknown stations. The algorithm generates preliminary coordinates independent of the order in which the data was collected, and can handle any combination of any type of loop traverse, link traverse, intersection, or resection. The preliminary coordinate generator tries to find closure routes as an aid to the user in blunder detection. Note in the above example two closure routes were found. BM1 and BM7 were not traversed through and thus are not on a closure route. They were observed from more than one station and are therefore redundant in nature. In that situation preliminary coordinates are generated for that station from one of the setups on which it was sighted.

A poor misclosure on one traverse can generate poor closures on subsequent traverses even though all of the subsequent traverses have good data. One should always look for a problem on the first poor traverse, and correct that problem and re-process before evaluating those subsequent traverses. The reason for this is best explained by example. A traverse from A1-A2-A3-A4-A5 where A1 and A5 are control points (or existing preliminary coordinates) reports a poor closure. The coordinates on a link traverse, which are always subjected to a compass rule adjustment at this stage, will result in poor coordinates due to the poor closure. A subsequent traverse A2-B1-B2-A3 could have good measurements but an apparent poor closure due to the end point's coordinates being generated from a bad traverse. The identification and correction of the problem in the first traverse link, followed by re-processing, will provide suitable coordinates for A2 and A3 from the first traverse, and thus suitable closures on the subsequent traverse.

8.7 Compass rule residuals

DISTANCE RESIDUALS

DISTANCE		RESIDUAL
A1	- A2	.016
A2	- BM1	.000
A2	- TRAV1	.008
TRAV1	- A3	.040
TRAV1	- BM1	.004
A3	- A2	.046
A3	- TRAV2	.021
TRAV2	- A2	.029
A3	- A4	.042
A3	- TRAV3	.024
TRAV3	- A4	.023
A4	- A5	.030
A4	- BM7	.000
A4	- TRAV4	.016
TRAV4	- A5	.014
TRAV4	- BM7	.013
TRAV4	- TRAV5	.017
TRAV5	- A4	.029
TRAV5	- A5	.001

ANGLE RESIDUALS

ANGLE			RESIDUAL (SEC)
A1	- A2	- BM1	.0
A1	- A2	- TRAV1	-3.1
A2	- TRAV1	- A3	.0
A2	- TRAV1	- BM1	-4.7
A2	- A3	- TRAV2	-1.8
A2	- TRAV2	- A3	.0
A4	- A3	- TRAV2	2.0
A4	- A3	- TRAV3	2.1
A3	- TRAV3	- A4	.0
TRAV3	- A4	- A5	-1.4
TRAV3	- A4	- BM7	.0
TRAV3	- A4	- TRAV4	-8.9
A4	- TRAV4	- A5	.0
A4	- TRAV4	- BM7	-15.0
A4	- TRAV4	- TRAV5	5.4
A4	- TRAV5	- A5	.0

When coordinate generation for every redundant station is complete, all redundant measurements are compared to the preliminary coordinates in generating pre-least squares residuals (the difference between what you measured and the value as derived from the coordinates. On measurements that existed on traverses the residual is due to the compass rule adjustment. In the case of unique solutions such as the angle-distance combinations for BM1 and BM7, the measurements used to compute that point's coordinates will have zero residuals as they were used in unique determination of those coordinates. The other measurements to BM1 and BM7 will contain pre-adjustment residuals as they were not used in the coordinate computation. If those measurements have large residuals, the problem could be in either the measurements used to determine the coordinates or in the ones that were not used.

Pre-adjustment residuals should be used with some care as their magnitude can be misleading. A series of traverse closures in a compass rule adjustment lead to error accumulation in the coordinates that compounds so later traverses and their measurements will appear of less quality than they possibly are. The simultaneous least squares adjustment, where there is no "hierarchy" of traverse routes, will allow this apparent error accumulation to be removed.

Chapter 9. Examination of processing's second report file - .1D

9.1 Example

The .1d file is the vertical least squares report. The 1726 station topo job's .1d file is shown in its entirety. Note this is derived from a different data set than the Chapter 8 example. This is a metric job.

```
MISCLOSURE OF MULTIPLE ELEV. DIFFERENCE MEASUREMENTS
  STATIONS          MISCLOSURE
A1       - A2             .005
A1       - A2             .010
A1       - A2             .008
A3       - TRAV2          .010
A3       - TRAV2          .000
A3       - TRAV2          .001
A3       - TRAV2          .008
A3       - TRAV2          .004
A3       - TRAV2          .008
TRAV3    - A4             .005
A3       - TRAV3          .026
TRAV3    - A4      *      .036*
A4       - TRAV4          .003
A4       - TRAV4          .005
A4       - TRAV4          .018
A4       - TRAV4          .012
A3       - TRAV3          .006
A3       - TRAV3          .002
END OF MISCLOSURE REPORT

 1708 OF  1726 STATIONS IDENTIFIED AS VERTICAL SIDESHOTS
BAND IS  10 STATIONS
LEVEL NETWORK ADJUSTMENT

NUMBER OF BENCHMARKS =     9
NUMBER OF STATIONS =    18
NUMBER OF MEASUREMENTS =    25
NUMBER OF REQUIRED TERMS FOR NORMAL EQUATIONS =      188

RESULTS OF ADJUSTMENT

BENCHMARK ELEVATION RESIDUALS

  STATION       INPUT ELEV.     ADJUSTED ELEV.     ERROR EST.     RESIDUAL

  A1                9.211            9.211         .001        .000  ( .0)
  BM1              10.101           10.101         .001        .000  ( .0)
  BM2               7.448            7.448         .001        .000  ( .0)
  BM3               9.114            9.114         .001        .000  ( .0)
  BM4               8.996            8.996         .001        .000  ( .0)
  BM5               8.898            8.898         .001        .000  ( .0)
  BM6               9.715            9.715         .001        .000  ( .0)
  BM7               9.717            9.717         .001        .000  ( .0)
  BM8               8.252            8.252         .001        .000  ( .0)

BENCHMARK RMS ERROR =       .000 SNOOP RMS =   .0
MAX. BENCHMARK RESIDUAL AT STATION         BM1      OF       .000


RESIDUALS

  FROM          TO            MEASURED       RESIDUAL      EST. ERROR

  A1            A2                .918          .003  ( .2)    .013
  A2            BM1              -.034          .003  ( .2)    .013
  A2            TRAV1            -.707          .005  ( .4)    .012
  TRAV1         A3               -.539          .043  ( .9)    .049
  TRAV1         BM1               .669          .002  ( .2)    .015
  A3            A2               1.172          .028  ( .5)    .058
  A3            TRAV2            -.115          .007  ( .6)    .013
  TRAV2         A2               1.274          .033  ( .9)    .039
  TRAV2         BM2             -1.374         -.003  ( .1)    .019
  TRAV2         BM3               .289          .000  ( .0)    .019
  A3            A4                .914          .068  (1.2)    .059
  A3            TRAV3            -.002         -.016  ( .9)    .017
  TRAV3         A4               1.008         -.007  ( .3)    .021
  TRAV3         BM4               .090         -.008  ( .3)    .030
  TRAV3         BM5              -.012         -.005  ( .4)    .013
  A4            A5              -1.455          .038  ( .8)    .049
  A4            BM6              -.201          .000  ( .0)    .018
```

```
  A4            BM7              -.202          .003  ( .1)    .022
  A4            TRAV4           -1.044          .001  ( .1)    .013
  TRAV4         A5               -.374          .000  ( .0)    .025
  TRAV4         BM7               .846         -.001  ( .1)    .015
  TRAV4         TRAV5            -.516          .006  ( .2)    .026
  TRAV5         A4               1.507          .046  (1.1)    .043
  TRAV5         A5                .138         -.002  ( .2)    .011
  TRAV2         BM8              -.584          .012  ( .6)    .020

ELEV. DIFF.  RMS ERROR =       .022 SNOOP RMS =   .5
MAX. ELEV. DIFF. RESIDUAL         A3       -         A4       OF       .068


95% CONFIDENCE F STATISTIC STANDARD ERROR MULTIPLIER FOR   16 D.F. IS  2.71

   STATION        ADJUSTED ELEV.     STANDARD ERROR

   A1                   9.211              .002
   A2                  10.132              .014
   BM1                 10.101              .002
   TRAV1                9.429              .019
   A3                   8.933              .022
   TRAV2                8.825              .018
   A4                   9.915              .018
   TRAV3                8.914              .018
   BM2                  7.448              .002
   BM3                  9.114              .002
   BM8                  8.252              .002
   A5                   8.498              .035
   BM6                  9.715              .002
   BM7                  9.717              .002
   TRAV4                8.872              .021
   TRAV5                8.362              .035
   BM4                  8.996              .002
   BM5                  8.898              .002


    STANDARD ERROR OF UNIT WEIGHT IS       .666
     WITH   16 DEGREES OF FREEDOM

CHI SQUARED TEST ON ANALYSIS
  .657 <   .666 <  1.282
  (LOW)           (HIGH)
PASSES AT THE 5 % SIGNIFICANCE LEVEL
```

Each section of the .1d report is now individually listed followed by a discussion of its information.

9.2 Compare same line on different setups

```
MISCLOSURE OF MULTIPLE ELEV. DIFFERENCE MEASUREMENTS
  STATIONS           MISCLOSURE
A1       - A2             .005
A1       - A2             .010
A1       - A2             .008
A3       - TRAV2          .010
A3       - TRAV2          .000
A3       - TRAV2          .001
A3       - TRAV2          .008
A3       - TRAV2          .004
A3       - TRAV2          .008
TRAV3    - A4             .005
A3       - TRAV3          .026
TRAV3    - A4       *     .036*
A4       - TRAV4          .003
A4       - TRAV4          .005
A4       - TRAV4          .018
A4       - TRAV4          .012
A3       - TRAV3          .006
A3       - TRAV3          .002
END OF MISCLOSURE REPORT
```

While elevation differences in the .gen report are compared to the first setup's values for that line, in this report the comparisons in this report are made to the existing weighted average for that elevation difference.

Note the tolerance was set at 0.03 m. for asterisks on elevation difference comparisons, and one appears larger than that. In this case this was not cause for alarm as it is on a long line (407 m.) which was being used as horizontal backsights, and the long line relates to a larger error estimate in the least squares analysis. Thus the long lines have fairly minimal weight in the least squares analysis. Some lines were measured many times as they served as backsights for several setups on the same point for large amounts of topographic data collection.

9.3 Number of sideshots, bandwidth, and adjustment network size

```
 1708 OF  1726 STATIONS IDENTIFIED AS VERTICAL SIDESHOTS
BAND IS  10 STATIONS
LEVEL NETWORK ADJUSTMENT

NUMBER OF BENCHMARKS =     9
NUMBER OF STATIONS =    18
NUMBER OF MEASUREMENTS =    25
NUMBER OF REQUIRED TERMS FOR NORMAL EQUATIONS =      188
```

Sideshots are non-benchmarks which are connected to only one other station. Spurs are removed by eliminating the legs of the spur(s) in an iterative fashion until all non-benchmarks are connected to at least two other stations. The surveyor is usually aware of approximately how many redundant elevation points should exist. Station misnaming is usually the problem when this number is incorrect. Note the number of horizontal sideshots may not equal the number of vertical sideshots as the location and number of horizontal vs. vertical control points could vary, and thus the redundant portion of the survey can contain different stations.

The band is an indicator of the size of the equations required for the vertical least squares analysis, and is thus an indicator to a user how long the solution will take to be resolved. The normal equations are the equations that are solved in least squares, and its number of terms is a function of the bandwidth and the number of stations. The time required for the solution to resolve is also a function of the number of stations and the processing speed of your computer.

The remaining information is simply some numeric information regarding the size of the survey. The number of measurements is reflected after weighted averaging has created one elevation difference for every line in the survey network.

9.4 Benchmark adjustment information

BENCHMARK ELEVATION RESIDUALS

STATION	INPUT ELEV.	ADJUSTED ELEV.	ERROR EST.	RESIDUAL
A1	9.211	9.211	.001	.000 (.0)
BM1	10.101	10.101	.001	.000 (.0)
BM2	7.448	7.448	.001	.000 (.0)
BM3	9.114	9.114	.001	.000 (.0)
BM4	8.996	8.996	.001	.000 (.0)
BM5	8.898	8.898	.001	.000 (.0)
BM6	9.715	9.715	.001	.000 (.0)
BM7	9.717	9.717	.001	.000 (.0)
BM8	8.252	8.252	.001	.000 (.0)

This portion reflects the benchmarks. Note through the use of error estimates in the .ctl file for benchmarks it is possible to allow them to be treated as a measurement in the adjustment instead of an absolutely fixed value. This would allow the benchmark elevations to adjust based on the least squares, and thus a residual (amount of adjustment) will appear. The 0.001 m error estimate reflects the benchmarks are to be held fixed (because 0.001 m is a very low error estimate compared to the error estimates on the elevation differences. The number in the parenthesis is the snoop number, which is the absolute value of the residual divided by the error estimate. Snoop numbers larger than 3.0 are asterisked as that reflects a high level of certainty that something is wrong with the data as the residual is more than three times larger than its error estimate. Note a large blunder can often create several other measurements to be asterisked. The definitions of snoop number, residual, and error estimate are consistent for all types of measurements in the .1d and .2d reports.

9.5 Benchmark rms errors and maximum benchmark residual

```
BENCHMARK RMS ERROR =       .000 SNOOP RMS =   .0
MAX. BENCHMARK RESIDUAL AT STATION         BM1      OF       .000
```

RMS is an abbreviation for root-mean-square. RMS error is the square root of the sum of the squares of the residuals divided by the number of measurements for that particular type of measurement. It can thus simply be described as a form of average residual. The snoop rms is the equivalent computation for the snoop numbers for that type of measurement. The maximum residual is the largest residual in an absolute sense (independent of error estimates and sign). Definitions of RMS error and maximum residual are consistent for all types of measurements in the .1d and .2d reports.

9.6 Elevation difference information

RESIDUALS

FROM	TO	MEASURED	RESIDUAL		EST. ERROR
A1	A2	.918	.003	(.2)	.013
A2	BM1	-.034	.003	(.2)	.013
A2	TRAV1	-.707	.005	(.4)	.012
TRAV1	A3	-.539	.043	(.9)	.049
TRAV1	BM1	.669	.002	(.2)	.015
A3	A2	1.172	.028	(.5)	.058
A3	TRAV2	-.115	.007	(.6)	.013
TRAV2	A2	1.274	.033	(.9)	.039
TRAV2	BM2	-1.374	-.003	(.1)	.019
TRAV2	BM3	.289	.000	(.0)	.019
A3	A4	.914	.068	(1.2)	.059
A3	TRAV3	-.002	-.016	(.9)	.017
TRAV3	A4	1.008	-.007	(.3)	.021
TRAV3	BM4	.090	-.008	(.3)	.030
TRAV3	BM5	-.012	-.005	(.4)	.013
A4	A5	-1.455	.038	(.8)	.049
A4	BM6	-.201	.000	(.0)	.018
A4	BM7	-.202	.003	(.1)	.022
A4	TRAV4	-1.044	.001	(.1)	.013
TRAV4	A5	-.374	.000	(.0)	.025
TRAV4	BM7	.846	-.001	(.1)	.015
TRAV4	TRAV5	-.516	.006	(.2)	.026
TRAV5	A4	1.507	.046	(1.1)	.043
TRAV5	A5	.138	-.002	(.2)	.011
TRAV2	BM8	-.584	.012	(.6)	.020

The elevation difference station names, measurements (weighted average for lines observed multiple times), elevation difference, residual, snoop number, and error estimate are presented. Everything is self-explanatory except for possibly error estimates. The general concept of error estimation is explained elsewhere in this user's guide. For elevation differences, the error estimate is generally derived from the repetition error (if measured more than once from the same setup), number of distinct measurements which have been averaged in a weighted procedure, and user defined constant and ppm (parts per million) error estimates. The ppm places larger error estimates on longer lines which is very logical in trigonometric leveling.

Of specific interest is that the magnitude of the asterisked values in the multiple elevation difference measurement comparison is not reflected as predominately in the least squares residuals. The weighted averaging process, especially when lines are measured in both directions, eliminates a major portion of this apparent misclosure.

9.7 Elevation difference rms errors and maximum elevation difference residual

```
 ELEV. DIFF.  RMS ERROR =       .022 SNOOP RMS =   .5
 MAX. ELEV. DIFF. RESIDUAL         A3       -        A4       OF       .068
```

These parameters are the same in nature for elevation differences as they were for benchmark elevations. Note the elevation difference (or any other) measurement RMS error does not reflect the difference in error estimates of measurements of that type. The snoop RMS is thus a better indicator of the overall relation of residuals to error estimates for that particular measurement type.

9.8 F statistic multiplier, adjusted elevations, and 95% elevation standard errors

95% CONFIDENCE F STATISTIC STANDARD ERROR MULTIPLIER FOR 16 D.F. IS 2.71

STATION	ADJUSTED ELEV.	STANDARD ERROR
A1	9.211	.002
A2	10.132	.014
BM1	10.101	.002
TRAV1	9.429	.019
A3	8.933	.022
TRAV2	8.825	.018
A4	9.915	.018
TRAV3	8.914	.018
BM2	7.448	.002
BM3	9.114	.002
BM8	8.252	.002
A5	8.498	.035
BM6	9.715	.002
BM7	9.717	.002
TRAV4	8.872	.021
TRAV5	8.362	.035
BM4	8.996	.002
BM5	8.898	.002

The F-statistic multiplier is applied to all one sigma standard errors of final coordinates in achieving 95 % confidence coordinate standard errors. As the redundancy (number of degrees of freedom) grows this multiplier decreases because added redundancy provides more confidence in your results. The adjustment's standard error of unit weight is also a multiplier in obtaining final coordinate standard errors.

Final adjusted coordinates and 95 % confidence standard errors (if this option is activated) follow. The meaning of the standard errors is that if you went and performed the same survey again under the same conditions you would be 95% confident you would be within the standard error of the adjusted elevation.

9.9 Standard error of unit weight and degrees of freedom

```
STANDARD ERROR OF UNIT WEIGHT IS       .666
 WITH   16 DEGREES OF FREEDOM
```

The standard error of unit weight is the square root of the sum of the snoop numbers squared divided by the number of degrees of freedom. If error estimation is being performed reasonably with regards to the quality of the measurements, and no blunders exist, the standard error of unit weight will be near one. Less than one indicates the measurements tend to be of better quality than the error estimates, and greater than one indicates the measurements are of less quality than the error estimates, or blunders exist. It is up to the surveyor to deem if the adjustment is valid through examination of standard error of unit weight, residuals, snoop numbers, and RMS errors.

The number of degrees of freedom is number of benchmarks plus number of elevation differences minus number of elevations. In our example those numbers are 9, 25, and 18 respectively, so the number of degrees of freedom is 9+25-18=16.

9.10 Chi squared test on standard error of unit weight

```
 CHI SQUARED TEST ON ANALYSIS
   .657 <   .666 <  1.282
   (LOW)            (HIGH)
 PASSES AT THE 5 % SIGNIFICANCE LEVEL
```

The chi squared test is a test of the validity of the least squares analysis, i.e., do the residuals reflect the quality of the error estimate based on the resultant standard error of unit weight. It is performed at the 5 % significance level which is 95% confidence. The high and low values of the range which defines passing the test shrinks as the number of degrees of freedom grows. This is because more redundancy should ensure more consistency in resultant data. Passing anything at 95% confidence is not easy, and thus one should not regard failure of the test as a need to reprocess the data. Use survey judgement in evaluation of residuals, snoop numbers, rms errors, and standard error of unit weight in verifying the suitability of the adjustment.

Chapter 10. Examination of processing's third report file - .2d

10.1 2D least squares input files - .lsa and .2sd

The .2d report is generated from two files - .lsa (measurements) and .2sd (error estimates). These files are generated by the abstracting and preliminary coordinate generation process of EFBP. The first four components of .lsa and .2sd complement one another. These sections (in order) are control coordinates, horizontal distances, horizontal angles, and azimuths. A line of data ends with a 0 or a 1. A 0 means more of that data type follows, while a 1 designates the end of that data type.

The control coordinate format is station X Y in .lsa, while in .2sd the X,Y error estimates replace the coordinates. The distances are from_station to_station distance (error estimate in .2sd). The angles are backsight_station occupied_station foresight_station degrees minutes seconds. Angle error in seconds replaces the measurement in .2sd. Azimuths are from_station to_station degrees minutes seconds. Azimuth error in seconds replaces the measurement in .2sd . All measurements are not yet reduced to grid if a datum has been selected. The fifth section of .lsa are the horizontal approximate coordinates required for the 2-d least squares to operate. The format is station_name X Y followed by a 0 or 1 as defined before.

The .lsa file for the presented .2d file follows. Note no azimuths exist in this job so the station names are zeroes.

```
A1          494886.316     128284.789    0
A2          495149.598     128318.648    0
A3          495980.147     128287.811    0
A4          496811.525     128276.865    0
A5          497318.565     128280.773    1
A1             A2                   265.467    0
A2             BM1                   51.774    0
A2             TRAV1                133.096    0
TRAV1          A3                   699.112    0
TRAV1          BM1                   82.310    0
A3             A2                   831.167    0
A3             TRAV2                349.099    0
TRAV2          A2                   482.199    0
A3             A4                   831.492    0
A3             TRAV3                424.092    0
TRAV3          A4                   407.406    0
A4             A5                   507.085    0
A4             BM7                  182.982    0
A4             TRAV4                269.433    0
TRAV4          A5                   237.796    0
TRAV4          BM7                   88.345    0
TRAV4          TRAV5                249.781    0
TRAV5          A4                   518.918    0
TRAV5          A5                    12.896    1
A1        A2        BM1        187 13 18.50 0
A1        A2        TRAV1      196  1 17.25 0
A2        TRAV1     A3         172 11  4.25 0
A2        TRAV1     BM1          5 31 17.00 0
A2        A3        TRAV2      358 49 46.25 0
A2        TRAV2     A3         177 58 58.75 0
A4        A3        TRAV2      180 12  9.25 0
A4        A3        TRAV3      359 55 14.50 0
A3        TRAV3     A4         180  9 47.03 0
TRAV3     A4        A5         178 43 13.88 0
TRAV3     A4        BM7        184 42  8.38 0
TRAV3     A4        TRAV4      179 59 58.13 0
A4        TRAV4     A5         177 16  2.87 0
A4        TRAV4     BM7        350 13 20.88 0
A4        TRAV4     TRAV5      176  8 55.88 0
```

```
A4          TRAV5       A5          336 52 17.12 1
0                 0                   0  0    .00   1
BM1                 495201.372          128318.744    0
TRAV1               495281.157          128298.531    0
TRAV2               495631.118          128293.637    0
TRAV3               496404.186          128282.819    0
BM7                 496993.652          128259.200    0
TRAV4               497080.913          128272.918    0
TRAV5               497330.332          128286.047    1
```

The .2sd file follows. Note the setup error for angles and azimuths is not yet applied to these values which explains why the error estimates in .2d are larger than the error estimates in this file. Note the shorter lines receive larger setup error contributions than longer lines.

```
A1               .001         .001    0
A2               .001         .001    0
A3               .001         .001    0
A4               .001         .001    0
A5               .001         .001    1
A1                A2                          .003    0
A2                BM1                         .005    0
A2                TRAV1                       .004    0
TRAV1             A3                          .009    0
TRAV1             BM1                         .006    0
A3                A2                          .010    0
A3                TRAV2                       .003    0
TRAV2             A2                          .007    0
A3                A4                          .010    0
A3                TRAV3                       .004    0
TRAV3             A4                          .004    0
A4                A5                          .008    0
A4                BM7                         .006    0
A4                TRAV4                       .003    0
TRAV4             A5                          .006    0
TRAV4             BM7                         .006    0
TRAV4             TRAV5                       .007    0
TRAV5             A4                          .008    0
TRAV5             A5                          .005    1
A1          A2          BM1            7.17 0
A1          A2          TRAV1          9.25 0
A2          TRAV1       A3             6.25 0
A2          TRAV1       BM1            8.00 0
A2          A3          TRAV2          7.25 0
A2          TRAV2       A3             7.25 0
A4          A3          TRAV2         10.75 0
A4          A3          TRAV3          7.50 0
A3          TRAV3       A4            18.47 0
TRAV3       A4          A5            10.75 0
TRAV3       A4          BM7            9.25 0
TRAV3       A4          TRAV4         11.50 0
A4          TRAV4       A5            10.87 0
A4          TRAV4       BM7            9.87 0
A4          TRAV4       TRAV5          9.88 0
A4          TRAV5       A5             7.13 1
```

10.2 Example Output

The .2d file is the horizontal least squares report. The 1726 station topo job's .2d file is shown in its entirety.

```
    PARAMETRIC HORIZONTAL LEAST SQUARES ADJUSTMENT

ALL MEASUREMENTS ARE REDUCED TO THE NAD 90
0903 FLORIDA NORTH LAMBERT

COORDINATE AND DISTANCE UNITS ARE METERS
BAND IS   5 STATIONS
NUMBER OF TERMS REQUIRED IN NORMAL EQUATIONS =     277

95% CONFIDENCE F STATISTIC STANDARD ERROR MULTIPLIER FOR   21 D.F. IS  2.64

                        RESULTS OF ADJUSTMENT

                                          STANDARD ERRORS  ERROR ELLIPSE INFO.
STATION            ADJUSTED X   ADJUSTED Y   IN X    IN Y     SU      SV     T

A1                 494886.316   128284.789   .001    .001   .001   .001   -7.3
A2                 495149.598   128318.648   .001    .001   .001   .001   -4.6
```

BM1	495201.370	128318.744	.005	.003	.005	.003	-86.5
TRAV1	495281.157	128298.531	.004	.004	.004	.004	-84.0
A3	495980.147	128287.811	.001	.001	.001	.001	1.4
TRAV2	495631.116	128293.637	.003	.007	.007	.003	1.2
A4	496811.525	128276.865	.001	.001	.001	.001	.1
TRAV3	496404.186	128282.823	.003	.010	.010	.003	.7
A5	497318.565	128280.773	.001	.001	.001	.001	-.4
BM7	496993.644	128259.196	.005	.006	.006	.005	19.0
TRAV4	497080.913	128272.921	.003	.005	.005	.003	.8
TRAV5	497330.333	128286.048	.004	.003	.004	.003	59.9

RESIDUALS IN THE OBSERVATIONS

CONTROL POINT COORDINATES

STATION	X RESIDUAL		X EST. ERROR	Y RESIDUAL		Y EST. ERROR
A1	.000	(.2)	.001	.000	(.0)	.001
A2	.000	(.0)	.001	.000	(.0)	.001
A3	.000	(.1)	.001	.000	(.0)	.001
A4	.000	(.1)	.001	.000	(.0)	.001
A5	.000	(.1)	.001	.000	(.0)	.001

EASTING CONTROL RMS = .000 SNOOP RMS = .1
MAX. EASTING RESIDUAL AT A1 OF .000
NORTHING CONTROL RMS = .000 SNOOP RMS = .0
MAX. NORTHING RESIDUAL AT A4 OF .000

DISTANCES

OCCUPIED STATION	SIGHTED STATION	DISTANCE	RESIDUAL		EST. ERROR
A1	A2	265.453	.002	(.6)	.003
A2	BM1	51.771	-.001	(.2)	.005
A2	TRAV1	133.089	.001	(.3)	.004
TRAV1	A3	699.075	.003	(.3)	.009
TRAV1	BM1	82.305	-.001	(.3)	.006
A3	A2	831.123	.002	(.2)	.010
A3	TRAV2	349.080	.001	(.3)	.003
TRAV2	A2	482.173	.006	(.8)	.007
A3	A4	831.448	-.002	(.2)	.010
A3	TRAV3	424.069	.001	(.3)	.004
TRAV3	A4	407.384	.001	(.4)	.004
A4	A5	507.058	.002	(.3)	.008
A4	BM7	182.972	-.002	(.3)	.006
A4	TRAV4	269.418	.002	(.5)	.003
TRAV4	A5	237.784	.002	(.3)	.006
TRAV4	BM7	88.340	-.002	(.3)	.006
TRAV4	TRAV5	249.768	.002	(.3)	.007
TRAV5	A4	518.890	.000	(.0)	.008
TRAV5	A5	12.895	-.001	(.3)	.005

DISTANCE RMS ERROR = .002 SNOOP RMS = .4
MAX. DISTANCE RESIDUAL TRAV2 - A2 OF .006

ANGLES

BACKSIGHT STATION	OCCUPIED STATION	FORESIGHT STATION	ANGLE	RESIDUAL (SECONDS)		EST. ERROR (SECONDS)
A1	A2	BM1	187-13-18.5	-1.8	(.2)	10.8
A1	A2	TRAV1	196- 1-17.2	-1.9	(.2)	9.9
A2	TRAV1	A3	172-11- 4.2	-1.3	(.2)	7.0
A2	TRAV1	BM1	5-31-17.0	-2.0	(.2)	9.9
A2	A3	TRAV2	358-49-46.2	-1.5	(.2)	7.4
A2	TRAV2	A3	177-58-58.7	.5	(.1)	7.4
A4	A3	TRAV2	180-12- 9.3	2.3	(.2)	10.8
A4	A3	TRAV3	359-55-14.5	3.7	(.5)	7.6
A3	TRAV3	A4	180- 9-47.0	-3.1	(.2)	18.5
TRAV3	A4	A5	178-43-13.9	.2	(.0)	10.8
TRAV3	A4	BM7	184-42- 8.4	-4.1	(.4)	9.6
TRAV3	A4	TRAV4	179-59-58.1	-4.9	(.4)	11.6
A4	TRAV4	A5	177-16- 2.9	-5.1	(.5)	11.1
A4	TRAV4	BM7	350-13-20.9	-3.3	(.3)	11.0
A4	TRAV4	TRAV5	176- 8-55.9	.8	(.1)	10.1
A4	TRAV5	A5	336-52-17.1	-1.8	(.1)	32.8

ANGLE RMS ERROR = 2.8 SECONDS SNOOP RMS = .3
MAXIMUM ANGLE RESIDUAL A4 - TRAV4 - A5
OF 5.1 SEC.

STANDARD ERROR OF UNIT WEIGHT IS .420
WITH 21 DEGREES OF FREEDOM
CHI SQUARED TEST ON ANALYSIS

```
  .700 <   .420 <  1.247
  (LOW)         (HIGH)
DOES NOT PASS AT THE 5 % SIGNIFICANCE LEVEL

---------------------------------------------------
TRAVERSE CLOSURE REPORT
SUM OF DISTANCES ALONG TRAVERSE IS        832.164
CLOSURE IN X =    -.004 CLOSURE IN Y =    -.004
ANGULAR CLOSURE =     1.3 SECONDS
LINEAR ERROR OF CLOSURE (AFTER ROTATION) IS         .004
BEFORE ROTATION PRECISION IS 1/      142855.
AFTER  ROTATION PRECISION IS 1/      216536.

STATION         BEARING       DISTANCE        X            Y

A2                                        495149.598   128318.648
TRAV1        S81-18-22.4E      133.088    495281.157   128298.531
A3           S89- 7-16.8E      699.073    495980.147   128287.811

---------------------------------------------------
TRAVERSE CLOSURE REPORT
SUM OF DISTANCES ALONG TRAVERSE IS        831.253
CLOSURE IN X =    -.007 CLOSURE IN Y =     .001
ANGULAR CLOSURE =     -.5 SECONDS
LINEAR ERROR OF CLOSURE (AFTER ROTATION) IS         .007
BEFORE ROTATION PRECISION IS 1/      125075.
AFTER  ROTATION PRECISION IS 1/      126100.

STATION         BEARING       DISTANCE        X            Y

A2                                        495149.598   128318.648
TRAV2        S87- 1-35.8E      482.168    495631.116   128293.637
A3           S89- 2-37.5E      349.079    495980.147   128287.811

---------------------------------------------------
TRAVERSE CLOSURE REPORT
SUM OF DISTANCES ALONG TRAVERSE IS        831.453
CLOSURE IN X =    -.003 CLOSURE IN Y =    -.006
ANGULAR CLOSURE =     3.1 SECONDS
LINEAR ERROR OF CLOSURE (AFTER ROTATION) IS         .003
BEFORE ROTATION PRECISION IS 1/      124017.
AFTER  ROTATION PRECISION IS 1/      323735.

STATION         BEARING       DISTANCE        X            Y

A3                                        495980.147   128287.811
TRAV3        S89-19-33.6E      424.068    496404.186   128282.823
A4           S89- 9-43.4E      407.382    496811.525   128276.865

---------------------------------------------------
TRAVERSE CLOSURE REPORT
SUM OF DISTANCES ALONG TRAVERSE IS        507.202
CLOSURE IN X =    -.003 CLOSURE IN Y =    -.006
ANGULAR CLOSURE =     5.1 SECONDS
LINEAR ERROR OF CLOSURE (AFTER ROTATION) IS         .003
BEFORE ROTATION PRECISION IS 1/       74806.
AFTER  ROTATION PRECISION IS 1/      153355.

STATION         BEARING       DISTANCE        X            Y

A4                                        496811.525   128276.865
TRAV4        S89- 9-40.5E      269.417    497080.913   128272.921
A5           N88- 6-27.5E      237.782    497318.565   128280.773

---------------------------------------------------
TRAVERSE CLOSURE REPORT
SUM OF DISTANCES ALONG TRAVERSE IS        531.785
CLOSURE IN X =    -.002 CLOSURE IN Y =     .000
ANGULAR CLOSURE =     1.8 SECONDS
LINEAR ERROR OF CLOSURE (AFTER ROTATION) IS         .002
BEFORE ROTATION PRECISION IS 1/      324257.
AFTER  ROTATION PRECISION IS 1/      335623.

STATION         BEARING       DISTANCE        X            Y

A4                                        496811.525   128276.865
TRAV5        N88-59- 9.6E      518.890    497330.333   128286.048
A5           S65-51-28.5W       12.896    497318.565   128280.773
```

```
TOTAL LENGTH OF EVALUATED TRAVERSE DISTANCE =      3.534 KM.

PRECISION BASED ON LATITUDE AND DEPARTURE CLOSURES = 1 /    128057.

PRECISION AFTER ORIENTATION CORRECTION = 1 /    197475.
```

The individual components of .2d are now discussed.

10.3 Datum parameters

```
 ALL MEASUREMENTS ARE REDUCED TO THE NAD 90
 0903 FLORIDA NORTH LAMBERT

 COORDINATE AND DISTANCE UNITS ARE METERS
```

This information is only output if a datum and zone are defined in the .ctl file using program CTL. EFBP only utilizes NAD 27 or NAD 83, but you are able to replace those numbers with a different year of adjustment. As an example, in Florida the High Accuracy Regional Network (HARN) was established/adjusted in 1990 and is known as NAD 83 (90). This survey utilized those coordinates and thus 90 was entered instead of 83. It is the same ellipsoid and state plane zone constants, but the 90 indicates to a reviewer which adjustment is referenced. A number larger than 82 tells EFBP to use NAD 83 constants, and a number less than 83 indicates to the program to use NAD 27 constants.

The state plane zone which is attributed in .ctl is output along with the units defined in the .ctl . In NAD 83 meters, survey foot, and international foot are the possible selections. In NAD 27 only survey foot is allowed. In assumed coordinates no units selection is output as no geodetic reductions are required. The .obs header is not used to defined units, the .ctl is used for this purpose.

The input files to the horizontal least squares are the .lsa (horizontal measurements) and .2sd (horizontal measurement error estimates files.

10.4 Bandwidth, adjustment size, and F statistic

```
 BAND IS   5 STATIONS
 NUMBER OF TERMS REQUIRED IN NORMAL EQUATIONS =      277

 95% CONFIDENCE F STATISTIC STANDARD ERROR MULTIPLIER FOR   21 D.F. IS  2.64
```

The definition of this information was described in the .1d file. It is the same for the .2d file as the definition is generic to any least squares adjustment.

10.5 Adjusted coordinates, 95% confidence coordinate standard errors and error ellipses

RESULTS OF ADJUSTMENT

			STANDARD ERRORS		ERROR ELLIPSE INFO.		
STATION	ADJUSTED X	ADJUSTED Y	IN X	IN Y	SU	SV	T
A1	494886.316	128284.789	.001	.001	.001	.001	-7.3
A2	495149.598	128318.648	.001	.001	.001	.001	-4.6
BM1	495201.370	128318.744	.005	.003	.005	.003	-86.5
TRAV1	495281.157	128298.531	.004	.004	.004	.004	-84.0
A3	495980.147	128287.811	.001	.001	.001	.001	1.4
TRAV2	495631.116	128293.637	.003	.007	.007	.003	1.2
A4	496811.525	128276.865	.001	.001	.001	.001	.1
TRAV3	496404.186	128282.823	.003	.010	.010	.003	.7
A5	497318.565	128280.773	.001	.001	.001	.001	-.4

BM7	496993.644	128259.196	.005	.006	.006	.005	19.0
TRAV4	497080.913	128272.921	.003	.005	.005	.003	.8
TRAV5	497330.333	128286.048	.004	.003	.004	.003	59.9

Station name, adjusted X (easting), and adjusted Y (northing) are always listed. The remaining items are 95% confidence error statistics if that option is turned on in the options menu of EFBP. Standard errors in X and Y mean if you did the same survey over under the same conditions you are 95% confident about the repeatability of that coordinate within that range. The size of these values grow as a point is further from control. An error ellipse is the same type of repeatability in a point's 2-D position. SU is the semi-major axis, SV is the semi-minor axis and T is the angle from north (clockwise positive) of the semi-major axis. Note a semi-distance is from the center of the ellipse to its exterior. An error ellipse follows the same logic as coordinate standard errors relative to distance from control.

10.6 Control point residual information

RESIDUALS IN THE OBSERVATIONS

CONTROL POINT COORDINATES

STATION	X RESIDUAL	X EST. ERROR	Y RESIDUAL	Y EST. ERROR
A1	.000 (.2)	.001	.000 (.0)	.001
A2	.000 (.0)	.001	.000 (.0)	.001
A3	.000 (.1)	.001	.000 (.0)	.001
A4	.000 (.1)	.001	.000 (.0)	.001
A5	.000 (.1)	.001	.000 (.0)	.001

EASTING CONTROL RMS = .000 SNOOP RMS = .1
MAX. EASTING RESIDUAL AT A1 OF .000
NORTHING CONTROL RMS = .000 SNOOP RMS = .0
MAX. NORTHING RESIDUAL AT A4 OF .000

Control error estimates, residuals, snoop numbers, and rms errors work the same as with benchmarks in the .1d report. Note northings and eastings are treated as separate measurements, and thus a northing control error estimate may not equal its easting partner. Traverse generally produces better coordinates in the cardinal direction in which the traverse runs.

10.7 Horizontal distance residual information

DISTANCES

OCCUPIED STATION	SIGHTED STATION	DISTANCE	RESIDUAL	EST. ERROR
A1	A2	265.453	.002 (.6)	.003
A2	BM1	51.771	-.001 (.2)	.005
A2	TRAV1	133.089	.001 (.3)	.004
TRAV1	A3	699.075	.003 (.3)	.009
TRAV1	BM1	82.305	-.001 (.3)	.006
A3	A2	831.123	.002 (.2)	.010
A3	TRAV2	349.080	.001 (.3)	.003
TRAV2	A2	482.173	.006 (.8)	.007
A3	A4	831.448	-.002 (.2)	.010
A3	TRAV3	424.069	.001 (.3)	.004
TRAV3	A4	407.384	.001 (.4)	.004
A4	A5	507.058	.002 (.3)	.008
A4	BM7	182.972	-.002 (.3)	.006
A4	TRAV4	269.418	.002 (.5)	.003
TRAV4	A5	237.784	.002 (.3)	.006
TRAV4	BM7	88.340	-.002 (.3)	.006
TRAV4	TRAV5	249.768	.002 (.3)	.007
TRAV5	A4	518.890	.000 (.0)	.008

TRAV5	A5	12.895	-.001	(.3)	.005

```
DISTANCE RMS ERROR =       .002 SNOOP RMS =   .4
MAX. DISTANCE RESIDUAL        TRAV2    -        A2       OF       .006
```

Distance measurement least squares residuals and related values operate the same as elevation differences in the .1d report. A very important additional point to note is that the distance reported is the measured value reduced to grid (or the measured value if an assumed coordinate system is designated). Only one distance (weighted mean) appears for each measured line no matter how many times it was measured in the .obs file. In the .1d file measured elevation differences (not adjusted values) are also listed. This theme of listed measured, not adjusted, values is consistent for all measurement types in the .1d and .2d reports. Residuals, snoop numbers, error estimates, and rms errors are similar to previous definition.

10.8 Horizontal angle residual information

ANGLES

BACKSIGHT STATION	OCCUPIED STATION	FORESIGHT STATION	ANGLE	RESIDUAL (SECONDS)		EST. ERROR (SECONDS)
A1	A2	BM1	187-13-18.5	-1.8	(.2)	10.8
A1	A2	TRAV1	196- 1-17.2	-1.9	(.2)	9.9
A2	TRAV1	A3	172-11- 4.2	-1.3	(.2)	7.0
A2	TRAV1	BM1	5-31-17.0	-2.0	(.2)	9.9
A2	A3	TRAV2	358-49-46.2	-1.5	(.2)	7.4
A2	TRAV2	A3	177-58-58.7	.5	(.1)	7.4
A4	A3	TRAV2	180-12- 9.3	2.3	(.2)	10.8
A4	A3	TRAV3	359-55-14.5	3.7	(.5)	7.6
A3	TRAV3	A4	180- 9-47.0	-3.1	(.2)	18.5
TRAV3	A4	A5	178-43-13.9	.2	(.0)	10.8
TRAV3	A4	BM7	184-42- 8.4	-4.1	(.4)	9.6
TRAV3	A4	TRAV4	179-59-58.1	-4.9	(.4)	11.6
A4	TRAV4	A5	177-16- 2.9	-5.1	(.5)	11.1
A4	TRAV4	BM7	350-13-20.9	-3.3	(.3)	11.0
A4	TRAV4	TRAV5	176- 8-55.9	.8	(.1)	10.1
A4	TRAV5	A5	336-52-17.1	-1.8	(.1)	32.8

```
ANGLE RMS ERROR =        2.8 SECONDS  SNOOP RMS =   .3
MAXIMUM ANGLE RESIDUAL        A4       -        TRAV4    -         A5
OF     5.1 SEC.
```

Angle residuals are again the difference between what is measured and the adjusted value. The measured value is the mean at a setup, thus another setup which measured that same angle would also appear. If a state plane zone was selected a small (usually less than 0.1 seconds) called the T-t correction is applied to all horizontal angles and would be reflected in the measured values. Note the lines usually have to be longer than 0.5 miles for the T-t correction to exceed 0.1 seconds. All residuals, error estimates, and rms errors are provided in seconds.

If azimuths were in a job they would follow the angles and have a similar format. Note azimuths do not exist in .obs but instead are in the control (.ctl) file. Astronomic azimuths in .ctl are reduced to grid and output in .2d as grid values. Error estimates are assigned to the azimuths during creation of the .ctl file.

10.9 Standard error of unit weight, degrees of freedom, and chi squared test

```
STANDARD ERROR OF UNIT WEIGHT IS      .420
```

```
    WITH   21 DEGREES OF FREEDOM
 CHI SQUARED TEST ON ANALYSIS
   .700 <   .420 <  1.247
   (LOW)            (HIGH)
 DOES NOT PASS AT THE 5 % SIGNIFICANCE LEVEL
```

The standard error of unit weight and the chi squared test on it are similar to its definition in the .1d report. In the .2d report the degrees of freedom is equal to 2 * the number of control coordinates + number of distances + number of angles + number of azimuths - 2 * total number of stations. This is more simply defined as number of measurements minus number of unknowns. Note the chi squared test "failed" on the good end - the residuals were smaller than the error estimates. While this concept of failure is statistically correct, as a surveyor one should simply be happy that this occurred and accept these results.

10.10 Traverse closure report

```
 ---------------------------------------------------
 TRAVERSE CLOSURE REPORT
 SUM OF DISTANCES ALONG TRAVERSE IS        507.202
 CLOSURE IN X =    -.003 CLOSURE IN Y =    -.006
 ANGULAR CLOSURE =     5.1 SECONDS
 LINEAR ERROR OF CLOSURE (AFTER ROTATION) IS         .003
 BEFORE ROTATION PRECISION IS 1/       74806.
 AFTER  ROTATION PRECISION IS 1/      153355.

 STATION         BEARING       DISTANCE        X            Y

 A4                                       496811.525   128276.865
 TRAV4        S89- 9-40.5E      269.417   497080.913   128272.921
 A5           N88- 6-27.5E      237.782   497318.565   128280.773
```

Traverse closures are performed after the least squares adjustment process is completed. Which traverse closures are performed in no way affects the least squares adjustment.

Only one traverse closure is shown as an example. EFBP tries to place every unknown redundant station on at least one traverse closure. Once an unknown point is on a closure route EFBP may choose to use its least squares adjusted coordinates as an end point of a link traverse. If you desire additional closure information one should run program CHK. Link traverses are computed between the adjusted coordinates of the traverse closure end points. The initial bearing is derived from the inverse of the adjusted coordinates of the first and second stations on the traverse. Measured distances and angles (reduced to grid if a datum is defined) are used in traversing to the end point. The closing bearing based on these computations is compared to that inversed between the end point and its connected station's least squares adjusted coordinates to obtain the angular closure on the link traverse.

Closure in easting (X) and northing (Y) is made by comparing the traverse based end coordinates to the least squares adjusted values. The linear error of closure after rotation is the comparison of the traverse based distance between end points compared to the distance based on adjusted coordinates. This distance comparison eliminates the rotation error due to the initial bearing assumption on the first line of the traverse.

The traverse length is the sum of the distances along the traverse. Both precisions use that sum of the distances in the denominator. The before rotation precision uses the closures in x and y in calculating the numerator misclosure. The after rotation precision uses the linear error of closure after rotation in the numerator.

Which is the "correct" precision? The after rotation precision is more indicative of the quality of your measurements as it is not based on the quality of the initial assumption of direction. A small error in initial direction will create a large error in northing and easting over a long distance simply due to how a few seconds of error would propagate over a few miles.

10.11 Total evaluated traverse network

```
TOTAL LENGTH OF EVALUATED TRAVERSE DISTANCE =      3.534 KM.

PRECISION BASED ON LATITUDE AND DEPARTURE CLOSURES = 1 /    128057.

PRECISION AFTER ORIENTATION CORRECTION = 1 /    197475.
```

The length of total evaluated traverse distance is provided along with overall precisions computed from sums of linear error of closures and traverse distances.

10.12 2D least squares other output files - .cor and .geo

In addition to the .2d report, the horizontal least squares outputs two adjusted coordinate files. The first is the .cor which are plane coordinates. A line in .cor is made up of station name, easting, northing, scale factor, convergence angle. If an assumed datum is used no scale factor or convergence angle exists. The .cor file of adjusted coordinates for this job is:

STATION	X COOR.	Y COOR.	SCALE FACTOR	CONVERGENCE
A1	494886.316	128284.789	.99994846	- 0-32- 54.0
A2	495149.598	128318.648	.99994846	- 0-32- 49.1
A3	495980.147	128287.811	.99994846	- 0-32- 33.5
A4	496811.525	128276.865	.99994846	- 0-32- 17.9
A5	497318.565	128280.773	.99994846	- 0-32- 8.3
BM1	495201.370	128318.744	.99994846	- 0-32- 48.1
BM7	496993.644	128259.196	.99994846	- 0-32- 14.4
TRAV1	495281.157	128298.531	.99994846	- 0-32- 46.6
TRAV2	495631.116	128293.637	.99994846	- 0-32- 40.0
TRAV3	496404.186	128282.823	.99994846	- 0-32- 25.5
TRAV4	497080.913	128272.921	.99994846	- 0-32- 12.8
TRAV5	497330.333	128286.048	.99994846	- 0-32- 8.1

The other produced adjusted coordinate file is the .geo file, which consists of station name, latitude, and longitude. If an assumed datum is used this file will be empty. The .cor and .geo files contain only stations which were in the 2-D least squares (no 2-D sideshots). The .geo file for this job is:

STATION	LATITUDE	LONGITUDE
A1	30- 9- 9.98215	85-35- 28.18986
A2	30- 9- 11.16345	85-35- 18.36345
A3	30- 9- 10.41849	85-34- 47.31596
A4	30- 9- 10.31770	85-34- 16.24461
A5	30- 9- 10.59894	85-33- 57.29854

BM1	30- 9- 11.18260	85-35- 16.42883
BM7	30- 9- 9.79942	85-34- 9.43286
TRAV1	30- 9- 10.55092	85-35- 13.44011
TRAV2	30- 9- 10.50015	85-35- .36084
TRAV3	30- 9- 10.38664	85-34- 31.46843
TRAV4	30- 9- 10.27171	85-34- 6.17655
TRAV5	30- 9- 10.77380	85-33- 56.86062

Chapter 11. Examination of final coordinate production - .XYZ and .SOE

After the 2-D least squares is completed, EFBP now computes coordinates for all horizontal sideshots (vertical sideshots were computed immediately after completion of the 1-D least squares adjustment). If a state plane zone and datum have been selected sideshots are also reduced to grid by appropriate elevation and scale factors.

The coordinate data is merged with the attribute information in .obs to form a final .xyz file. If any station-offset or rod reading (SOR, SO, or R) records exist in .obs a .soe file will also be generated. If one has comments (O 99 records) in .obs these lines are transferred by EFBP to the .xyz and .soe files. In this example data some comments existed in .obs and were thus translated to the .xyz file.

The .xyz file is very column specific for station names (left justified), attribute information, coordinates (decimal point column location), and coordinate standard errors (decimal point column location). The line with all numbers in it is not part of a .xyz file, but is merely shown to better define column location. Since EXAMP was an 18 station job, only a part of its .xyz is shown to highlight all of its components.

```
123456789112345678921234567893123456789412345678951234567896123456789712
G 00 EXAMP.XYZ
G 01 A1       46-93-GPS2       P F1 10-6"_ROUND_FDOT          0903 90M88
G 02   494886.316   128284.789      9.211      .001     .001     .002
G 01 A2       46-93-E05        P G1 11-4"X4"_FDOT_CONC        0903 90M88
G 02   495149.598   128318.648     10.132      .001     .001     .014
G 01 A3       46-93-E06        P F1 11-4"x4"FDOT_CONC         0903 90M88
G 02   495980.147   128287.811      8.933      .001     .001     .022
G 01 A4       46-93-E07H       P F1 12-5/8COP.COT.ROD         0903 90M88
G 02   496811.525   128276.865      9.915      .001     .001     .018
G 01 A5       46-93-E08H       P F1 13-10_SPIKE_NAIL          0903 90M88
G 02   497318.565   128280.773      8.498      .001     .001     .035
G 01 AP1                       P G1                           0903 90M88
G 02   494951.660   128332.445      9.822
G 01 AP10                      P G1                           0903 90M88
G 02   494986.242   128320.241      9.830
G 01 BC13                      P G1                           0903 90M88
G 02   494986.122   128320.215      9.830
G 01 BC2                       C G1                           0903 90M88
G 02   494901.537   128335.450      9.166
G 01 BC3                       C G1                           0903 90M88
G 02   494904.200   128327.247      9.112
G 01 BC4                       C G1                           0903 90M88
G 02   494908.390   128323.249      9.097
G 01 BC5                       C G1                           0903 90M88
G 02   494915.627   128321.088      9.084
G 01 BC6                       P G1                           0903 90M88
G 02   494931.979   128320.866      9.195
G 01 MH1                       P G1 59-0.61M_                 0903 90M88
G 02   494918.576   128322.199      9.151
G 01 MH2                       P G1 59-.61_SEWER              0903 90M88
G 02   494902.200   128322.399      8.991
G 01 MH3                       C G1 59-.61                    0903 90M88
G 02   494971.282   128321.497      9.720
G 01 SIGN1                     P G1 26-US98_E&W               0903 90M88
G 02   494915.723   128322.343      9.168
G 01 SIGN2                     P G1 26                        0903 90M88
G 99 COOK WHITEHEAD USED CARS .244 SQUARE METAL POLE
G 02   494908.246   128327.600      9.456
G 01 SIGN3                     P G1 26-US98                   0903 90M88
G 02   494903.155   128340.749      9.297
G 01 SIGN14                    P G1 26                        0903 90M88
G 99 SLIPPERY WHEN WET
G 02   495029.606   128299.221     10.067
G 01 SIGN15                    P G1 26                        0903 90M88
G 99 DIVIDED HIWAY ENDS
G 02   495107.892   128297.774     10.099
G 01 SIGN16                    P G1 26-2_LANE_HIWAY           0903 90M88
```

```
G 02   495277.446   128295.986      9.324
G 01 SIGN17                   C G1 26-SPEED_LIMIT_45        0903 90M88
G 02   495150.807   128316.440     10.133
G 01 TREE2                    P G1 50-.244_PALM             0903 90M88
G 02   495843.061   128276.975      9.026
G 01 TREE3                    P G1 50-.3048_PALM            0903 90M88
G 02   495846.543   128279.170      9.174
```

The G 00 record is the header of the file and really only serves to ensure the validity of the project name and file purpose. Each coordinate record always consists of a G 01 and a G 02 record. If comments are attached to the point, these line(s) show as a G 99 record and are in between the G 01 and G 02 record for the point.

The G 01 record contains the point name (up to eight alphanumeric characters with no spaces), reference name (optional in columns 15-30) geometry (point-P vs. curve-C), attribute (ground-G, feature-F, or user defined-U), zone number, feature code, short description (after a dash which follows a feature code, state plane NGS zone number (4 digit integer), datum/adjustment year (2 digit integer), coordinate units (M, F, or I), and vertical datum (2 digit integer). A lack of state plane zone or datum/adjustment year indicates assumed coordinates. What fields must absolutely be filled is a function of the software to which this information will be loaded and has no affect on EFBP except for the facts that EFBP is station name based and a zone and datum/adjustment year are required for geodetic processing. This example was in 0903 - Florida North Lambert adjustment year 1990 so the ellipsoid being used is from 1983. The coordinates are metric, and the elevation datum is North American Vertical Datum (NAVD) of 1988.

The G 02 record contains X (easting), Y (northing), Z (elevation), X standard deviation, Y standard deviation, and Z standard deviation.
The standard deviations are the 95 % confidence values from the .1d and .2d files. These are not determined for sideshots, and are only determined if that computational option is turned on in the EFBP initial options.

The G 99 record exists if any extended comments are entered about a point. In this job one can see the message on road signs were entered as comment records. Some sign messages which were short were entered as short descriptions attached to the feature with the "-" option.

If a station only has horizontal coordinates the Z area will be blank. Likewise, the X,Y areas are blank if a station has elevation only.

This example did not contain SOR (station, offset, rod reading/leveling) type data so an .soe (station, offset, elevation) file was not created. A portion of a SOE file is shown to demonstrate its content. Note elevations from differential leveling will also appear in .xyz, but no horizontal coordinates are calculated by EFBP for station offset as it needs a calculated alignment which does not usually exist at this point.
The file demonstrates station-offset-elevation, station-offset only, or elevation only (differential leveling). The numbered line is simply shown as the file is column specific similar to the .xyz file.

```
123456789112345678921234567893123456789412345678951234567896123456789712
G 00 KOGER.SOE
G 01 B6       LEO-115-FLDNR    P G1 17-ELEV.203.67FT
G 04                                               203.670     .002
G 01 B7       TP1              P G1
G 04                                               213.691     .044
G 01 B21      LEO-76-FLDNR     P G1 17-ELEV.169.94FT
G 04                                               169.940     .002
G 01 D1                        P G1 50                      XS1
G 04              1025.00   75.50
G 01 D10                       P G1 50                      XS1
G 04              1100.00  175.50
G 01 D11                       P G1 50                      XS1
G 04              1110.00  201.50
G 01 D12                       P G1 50                      XS1
G 04              1110.00 -140.60
G 01 D13                       P G1 50                      XS1
G 04              1120.00  -50.00
G 01 D14                       P G1 50                      XS1
G 04              1150.00   51.50
G 01 D15                       P G1 50                      XS1
G 04              1165.00 -321.60
G 01 D16                       P G1 50                      XS1
G 04              1170.23   23.25
G 01 D17                       P G1 23                      XS1
G 04              1023.00   51.00                  211.684
G 01 D18                       P G1 24                      XS1
G 04              1046.20  -61.29                  212.634
G 01 D19                       P G1 25                      XS1
G 04              1046.20  -51.00                  211.584
G 01 D2                        P G1 50                      XS1
G 04              1035.00  -25.00
G 01 D20                       P G1 26                      XS1
G 04              1126.00  -15.00                  212.084
G 01 D3                        P G1 50                      XS1
G 04              1040.00  -35.50
G 01 D4                        P G1 50                      XS1
G 04              1050.00  202.50
G 01 D5                        P G1 50                      XS1
G 04              1060.00 -205.50
G 01 D6                        P G1 50                      XS1
G 04              1070.50  199.80
G 01 D7                        P G1 50                      XS1
G 04              1080.00 -210.00
G 01 D8                        P G1 50                      XS1
G 04              1090.00  155.50
G 01 D9                        P G1 50                      XS1
G 04              1095.00  -50.50
G 01 D21                       P G1 26                      XS1
G 04              1050.00  200.50                  213.554
G 01 D22                       P G1 26                      XS1
G 04              1100.00 -124.50                  203.614
G 01 D23                       P G1 26                      XS1
G 04              1150.00  199.50                  215.634
```

The G 00 is simply a header record which defines the nature of the file. For each point a G 01 and G 04 record always exists, and G 99 records will be between these lines if comments exist for some points in the .obs file.

The G 01 record consists of station name, reference name (very optional), geometry, attribute, zone number, feature code, feature code description, and alignment geometry chain name. Which fields are optional is a function of the software reading this file, and not EFBP which produces it. The feature code description is optional and requires a dash after the feature code. No alignment name is required for stations with only elevations as there is no way to horizontally position them without station-offset or horizontal coordinates.

The G 04 record consists of station, offset, elevation, and elevation standard deviation. The standard deviations are the 95 % confidence values from the .1d file. These are not determined for sideshots, and are only determined if that computational option is turned on in the EFBP initial options.

Fields are left blank if that information was not obtained. Note B7 only has elevation, while D4 has only station offset. Stationing and offset are in units of feet or meters, and a negative offset indicates a left offset while facing the positive direction of stationing.

Chapter 12. Utility programs

These utility programs are used fairly infrequently but still provide a valuable service to the surveyor using EFBP.

Computer Program LOOK

This program is for quick viewing of report and data files. It reads TEMP.JOB and displays all of the project files. You simply use the up-down keys to pick which file you want to look at. The program has search and printing routines built into it. Hitting escape gets you out of the program or out of a file. It is not quite as flexible as a text editor but you do not have to type a file name in if you use it!!!!!!!!! You will probably still find your text editor more flexible for cleaning up the .obs file especially if you view .obs and .gen simultaneously.

Computer Program CHK

This program outputs to .CHK and lets you compute any closure you want. You can also hold unknown station's adjusted coordinates as control to force a closure into it. The closure's are 3-D if the third dimension has closure in it.

Computer Program LCHK

This program outputs to .CHL and lets you compute any vertical closure you want. You can also hold unknown station's adjusted coordinates as control to force a closure into it. It handles both trig. leveling and differential leveling. If the data is 3 wire or trig. leveling you get a precision based on the distance traversed or levelled (assuming a stadia constant in leveling of 100). If you have lots of leveling not in EFB a program called 3wire exists which will process 1, 2, or 3 wires for you from data in a simple text file.

Computer Program FIXIT

FIXIT allows you to alter slope distances, height of instruments, and height of targets by scale and constant values. This is useful for cleaning up a prism constant error or converting entire English .obs files from feet to metric or vice-versa. It has been used to clean up decimal point problems that EFB in the field created. Station, offset, and level rod readings can also be corrected for scale and constant errors. Note eccentrics are not corrected by FIXIT.

Chapter 13. Understanding Least Squares, Adjustment, and Error Estimation - A Basic Approach for Utilization

13.1 Introduction

The use of adjustment of survey measurements is obviously something the surveying community wishes could be avoided. Unfortunately we, and the instrumentation we use, are not perfect in our measuring ability, and adjustment is the term that has been associated with the procedures we use in accounting for our inconsistencies.

If an adjustment changes a measurement within an acceptable random error limit we should not be concerned. The statistical difference between the adjusted and the measured quantity in this situation is negligible. A 4 second adjustment of a horizontal angle measured twice with a 6 second least count theodolite is obviously within the expected error range of the angle. A 30 second adjustment of that angle could easily be termed intolerable, and a surveyor should not accept the adjustment. The cause of the intolerable adjustment (data entry error, field blunder, etc.) needs to be determined and the measurement corrected. The readjustment based upon the correction then needs to be evaluated for acceptability. One role of this paper is to explain why adjustment can be a valid procedure, and how to judge when it is not valid.

A large number of the surveying community are familiar with the compass rule as an effective adjustment procedure for a loop traverse adjustment. Unfortunately, this approach is limited in versatility when a traverse network (series of interconnected traverses) is encountered. A least squares approach to adjustment of survey data analyzes any survey network configuration in the same fashion. Least squares analysis is not limited to surveying. It is an accepted procedure in mathematics, statistics, computer science, and a variety of engineering disciplines. In most other disciplines least squares is considered a "data analysis" technique as opposed to an adjustment process. This paper will illustrate that it is really an analysis technique in surveying, too. Adjustment actually exists in any redundant survey network - misclosures are simple restricted to a limited number of closing measurements in an "unadjusted" situation. While this unadjusted approach may be a valid procedure in some cases, one would normally desire a more uniform adjustment procedure since we know this is how random errors occur in our measurements. No matter what technique is used, if redundancy exists it will be shown that adjustment exists.

The final point to be addressed in this paper is the accepted lack of understanding of least squares analysis in a large component of the surveying population. This is partially due to the lack of understandable reading material on the subject, and that it requires a computer for it to be implemented in a production environment. The personal computer revolution is a recent phenomena which has allowed a surveyor access to least squares for survey network analysis, and efficient PC based software is no longer a difficult commodity to locate.

One does not require an in-depth understanding of linear algebra, calculus, statistics, or computer science to become a knowledgeable user of least squares analysis. A computer, dedicated software, and a few hours of hands-on instruction and use are required. This paper does not replace hands-on instruction and use, but can serve as a primer for those who are unfamiliar with this approach. This paper is directed to a user, and not to one who wishes to understand the mathematics which is being utilized in least squares operations.

13.2 Is There Really Such a Thing as "Unadjusted" Data?

The answer to this question is obviously "yes", but it pertains only to non-redundant data or the raw measurements themselves. Once redundant information such as a loop traverse is created there is always some form of adjusted information, even if an adjustment procedure such as the compass rule is not applied.

A three-sided loop traverse provides a very simple example of this phenomena. Assume all interior angles have been measured along with the three distances, station 1 has fixed coordinates, and the direction from 1 to 2 is known.

```
      2  /\
        /  \
       /    \
      /      \
  1  /________\ 3
```

The assumption of coordinates at 1 and the fixed direction "fixes" the network's position and orientation in a 2-D coordinate system.

Let us first consider only angles. It is known that the sum of the interior angles of an n-sided polygon must equal (n-2)*180 degrees, and due to random error it is unlikely that the angles in the example will sum to that. If you treat two of your angles as "unadjusted", by the defined angular geometry [(n-2)*180] the third angle is automatically adjusted by -1 * the angular closure. If one of the angles was left unmeasured, there would be no redundancy in angles and thus no logical way to adjust angles.

If one assumes a "raw" closure (no angle adjustment prior to linear closure computation) is desired, in an n-sided polygon there are always n possible linear closures that can be computed. In the three sided example a linear error of closure can be computed beginning and ending at station 1 without using the angle at station 1. The same procedure can begin at station 2 and station 3, in each case not using the angles at 2 and 3 respectively. In each case a different linear closure will be realized because the same measurements are not used in all three cases. There are thus three unique compass rule adjustments possible if angles are not adjusted prior to closure computation.

Even if this confusing issue is ignored, a loop traverse with a non-zero linear closure has to have adjustment due to the geometric constraint of sum of latitudes and departures each totalling zero. Simply computing coordinates of 2 and 3 clockwise from station 1 will ignore use of the angles at 1 and 3 and the distance from 2 to 3. This omission actually places all adjustment in these three measurements.

As stated before, one should observe if the amounts of angular and distance adjustment are within reasonable random error limits. If this is true, the amounts of adjustment are within the limits of one's measuring abilities. If the amount of adjustment to various measurements is not within acceptable limits the adjustment process is indeed invalid. The source of the unreliable measurements should be determined and corrected.

This demonstrates that if redundancy exists adjustment has to exist. If redundancy exists geometric constraints prevents all measurements from remaining unadjusted.

13.3 When does a Compass Rule Approach to Traverse Adjustment become Difficult?

A compass rule adjustment of a loop traverse with reasonable angular and distance closures is a very valid procedure, and will often produce final coordinates which are very close to those produced by a least squares adjustment. Unfortunately a loop traverse is not a highly redundant situation. A loop traverse is also fairly weak geometrically, and can have undetected larger compensating errors in it. The sum of the interior angles, sum of latitudes, and sum of departures are the only geometric constraints in a loop traverse. It can also be shown by a mathematical technique called error propagation that a traverse's geometry is strongest in the direction it "runs". A section of traverse running north-south will therefore have weaker easting and stronger northing coordinates. Consider the north-south traverse as a guitar string. It bends easier east-west than it can be stretched north-south. Now let us assume another traverse section is surveyed in an east-west direction and it intersects the north-south section. If coordinates have already been computed on the north-south line (and possibly adjusted) the east-west section will be tying into coordinates weak in easting. If you compute coordinates on the east-west section first the north-south section will be intersecting weak northing coordinates. Least squares provides the solution to this problem.

Before we discuss why a least squares approach resolves this problem, let us consider another problem involved in compass rule type adjustment of a complicated traverse network. This problem involves priority of the lines in the adjustment process. Let us assume the following traverse network, with each side of the polygons representing series of angle and distance measurements.

```
                    2
       _________________________________
       l               l                l
       l               l                l
       l               l                l
       l               l                l
       l               l                l
     3 l_______________l_5______________l 1
       l               l                l
       l               l                l
       l               l                l
       l               l                l
       l               l                l
       ---------------------------------
                    4
```

Which of the following priorities of adjustment should be used:

```
Case A:  (1)  outside loop 4-1-2-3
         (2)  2-5-4
         (3)  3-5
         (4)  1-5

Case B:  (1)  outside loop 4-1-2-3
         (2)  3-5-1
         (3)  4-5
         (4)  2-5

Case C:  (1)  3-4-5-3
         (2)  4-1-2-3
         (3)  2-5
         (4)  1-5

Case D:  (1)  1-5-4-1
         (2)  5-3-4
         (3)  5-2-3
         (4)  2-1
```

etc., etc., etc.

There is an incredibly large number of adjustment priority combinations, with little or no ability to judge which priority is "best". Consider how many possibilities exist in a USPLS township where all township (exterior) and section lines have been traversed.

A common "rule of thumb" procedure in compass rule approaches to traverse networks is to adjust the exterior of the network as one loop first, then decide on a priority approach of adjusting interior traverses to "fit" the adjusted exterior. This concept is often termed a rigid boundary approach. This technique is the worst possible approach that could be used from an elementary error propagation standpoint. The exterior will generally contain more stations than any other loop in the network, and thus you would expect generally worse closures in it than any smaller loops. You would thus be adjusting the component with the largest expected error, and forcing smaller traverse sections (less propagated error) to fit it.

There is no statistically correct procedure that defines sequential compass rule adjustments of a traverse network. As opposed to a sequential procedure, least squares allows simultaneous adjustment of any traverse network geometry (including any number of control stations). This will initially seem impossible since many surveyors are not familiar with how least squares works, and thus a basic discussion of concepts is now required.

13.4 What does one need to know to feel Comfortable about Using Least Squares Analysis?

Whenever we measure something more than once and average the repetitions a least squares solution is being performed. An average minimizes the sum of the squares of the residuals. This is the underlying principle of least squares. A residual is the difference between the measurement and its adjusted quantity (an adjusted quantity being a simple average in this case). The residuals have to be squared because they tend to be both positive and negative, and thus a simple summation produces zero if a simple average is being computed.

Now that one realizes he or she has been using least squares all along in averaging, we now need to define how it applies to survey networks, and where the concept of weighting (or error estimation) of survey measurements becomes important. Horizontal survey networks consist of measured azimuths, angles, and distances in addition to control coordinates. Least squares adjusts all measurement types simultaneously in addition to adjusting all traverse legs simultaneously. This means relating angular quantities (azimuths and angles) to those of linear dimensions (distances and coordinates). To allow this, the least squares condition is expanded to minimizing the sum of the squares of the weighted residuals. A weight is equal to one divided by the measurement's error estimate.

Adding the weight into the least squares condition makes all terms in the summation unitless. As an example, a distance, its error estimate, and its residual, all have units in feet. (1 /error estimate) * residual is equal to (1 / feet) * feet in unit terms so the determined quantity becomes unitless. The unitless quantity can be directly compared to unitless quantities derived from angles and azimuths.

Notice that the least squares condition has no dependency on number of traverse legs, number of traverse connections, number of measured azimuths, number of control points, etc. At this time it is also illustrated that the least squares principle can be applied to any measurement system in any science (not just surveying). It can also be applied to differential leveling, 3-D traversing, GPS vectors, or any combination of survey measurements.

One estimates errors in various measurements using knowledge and experience. While repetition of a measurement such as an angle gives a clue to its reliability, the error estimate derived from repeated measurements does not usually take into account instrument and target setup errors, some atmospheric errors, etc. In other words, the standard error computed from a series of repetitions often tends to be smaller than what a surveyor should estimate since it does not account for several possible error sources. An obvious example is repeated EDM measurements from the same instrument / target setup. Often these values will not differ by more than 0.01 ft. but the combined error in instrument and reflector setups over their respective points can easily be larger than this number.

Angle and azimuth error estimates are a function of least count of an instrument, number of repetitions, and stability of instrument setup among other factors.

An interesting option of least squares is that control coordinates can be assigned error estimates and allowed to adjust. If control coordinates are not to adjust, they should be given a very small error estimate (such as 0.001 ft.). The control coordinate can therefore be treated as a measurement in the least squares solution.

Least squares has procedures which allow one to verify if one's error estimates are reasonable. The first simple check is a look at all of the measurement residuals (amount of adjustment applied to each measurement) and see if any are much larger in magnitude (absolute value) than the error estimate. If all but a few are acceptable, the unacceptable ones are obviously possible blunders. If a majority are much smaller than your error estimates you have been pessimistic about the quality of your work, or there is very little redundancy in your work.

To scan thousands of residuals would be extremely tedious so two "global" error computations can be made. The first is known as a root-mean-square (RMS) error. For a particular measurement type, RMS error is the square root of the sum of the squared residuals, divided by the number of that type of measurement. It could be thought of as an average residual for that particular type of measurement. The RMS error should be near in magnitude to your average error estimate for a particular type of measurement.

A value which is an indicator of your error estimating abilities for all types of measurements (the entire survey network) is the standard error of unit weight. To compute this quantity the sum of the squares of the weighted residuals is divided by the number of redundant measurements (termed number of degrees of freedom), and the square root of that computed quantity is taken. Since a residual and its respective error estimate should be near equal in magnitude, a weighted residual tends to be near one in amount. Of course, random error says some will be larger than one and some smaller than one. If this is true the standard error of unit weight will tend to be near one. A value larger than 2.5 indicates error estimates were optimistic or blunders exist. A value less than 0.7 indicates pessimistic error estimates or simply a lack of redundancy (therefore no adjustment of measurements is possible). A test can be applied to the standard error of unit weight to statistically validate the quality of your error estimates. A "rule-of-thumb" approach of 0.7 to 2.5 works well, too. A failure of this test indicates an invalid adjustment due to poor error estimates or blunders. If no blunders can be found, it is generally appropriate to change some or all of your error estimates.

Notice you can use your error estimates to "classify" different qualities of surveys within a network, and adjust all of the network simultaneously. Least squares still preserves the geometric constraints of sum of interior angles being (n-2)*180 degrees and sum of latitudes and departures of a loop being zero. Adjusted angles will close perfectly between adjusted azimuth measurements. If the data is azimuths (or bearings) and distances with no angles the sum of the interior angles constraint disappears but the latitude and departure constraints remain. This is no different than information derived from a compass rule approach. A compass rule adjustment actually produces residuals for all measurements, but they are generally not termed residuals. The simultaneous analysis of all data on all traverse legs, with the ability to weight measurements using your error estimates, is the key difference.

The simultaneous approach allows the systematic distortions which result from a sequential compass rule approach to be eliminated. You truly obtain the "best" coordinates based on a series of measurements and your error estimates.

13.5 Additional Information about Least Squares Adjustment of Horizontal Survey Data

A brief discussion of items involved in least squares analysis of horizontal survey data will help users with questions like "Why do I do this?" and "What do these numbers represent?". This will be presented in a question and answer format.

(1) Why does adjustment of large networks take so long to run?

Least squares analysis of survey networks requires solution of a system of equations equal in size to the number of unknowns. Unknowns in horizontal survey networks are the 2-D coordinates of the stations. A 1000 station network has 2000 unknown coordinates (1000 N, 1000 E). This means a system of 2000 equations, 2000 unknowns (called the normal equations) needs to be solved. Even on a fast computer this is not a trivial process. As the size of the network increases the time required for the solution increases exponentially (i.e., a 1000 station network takes more than twice the time to solve as a 500 station network).

(2) Why does least squares of horizontal networks require an iterative solution?

The equations used in horizontal data are "non-linear" equations. The inverse distance and azimuth equations are examples of non-linear equations because they include items such as square roots, squared terms, and trigonometric functions in them. A leveling network produces linear equations of the form:

Elev. B - Elev. A = meas. elev. difference + residual

There are no powers or trigonometric functions in this equation. To solve non-linear equations the equations are so-called "linearized". In doing this one creates a solution which solves for updates to approximations for all unknowns. The updates are added to the approximations, and the solution for the updates occur again (the second iteration). When the updates become "negligibly small" (all less than 0.005 ft. for most surveying applications) the solution has converged on the least squares solution for all coordinates.

Several questions arise:

(a) How are approximations for all coordinates generated, and how close to their final adjusted values do they have to be?

Conventional traverse computations is the most effective approach to approximate coordinate generation. This has been implemented for any traverse network configuration in computer program EFB of the Electronic Field Book Processing (EFBP) software package developed by the author. EFB performs a compass rule adjustment for all traverse routes, outputs closure reports and residuals for all measurements from the compass rule. These serve as excellent approximations unless blunders exist, and are generated automatically from any unordered data set.

(b) How many iterations are usually required?

If no blunders exist the solution will usually terminate in 2 iterations. The first iteration will illustrate the difference between the compass rule and least squares solution, and the second iteration will produce all negligible updates.

If blunders exist a solution will often run more than 2 iterations. There is generally little need to run more than 4 iterations for a data set.

(c) What is a divergent solution?

If approximations are very poor, or substantial blunders exist, it is possible that iteration updates will grow (diverge) instead of decrease (converge). It is important that one looks at pre-adjustment closures and compass rule residuals (produced by EFB) for obvious blunders, which can be corrected, prior to running the least squares adjustment.

(3) What are the meanings of standard errors of coordinates and error ellipses?

These quantities give the surveyor a feel for the positional reliability of his or her produced coordinates. They are a function of the proximity of the coordinates to control stations in the network (further from control you are less confident of the positional reliability of your coordinates). These error analysis results are very computationally intensive to obtain, and thus should only be generated for large networks if truly necessary.

A short discussion of statistical properties of standard errors is required to understand this component of least squares results. Least squares analysis assumes measurements are drawn from a normal distribution. A normal distribution results in the familiar bell-shaped curve of statistics. Values of data is represented on the x axis and frequency of occurrences on the y axis. The center of the bell shaped curve is the average. Statistics can show that approximately 66.7% of your data falls within the region between the average minus the standard error and the average plus the standard error.

This applies directly to standard errors of survey coordinates. If you performed the same survey over, using the same techniques under the same conditions, you would be 66.7% confident that the new coordinate will fall within "plus or minus" the standard error of the first produced coordinate. The error ellipse produces SU - maximum error, SV - minimum error, and T - angle from north to the direction of the maximum error direction. One will normally find maximum error approximately 90 degrees from the direction of the traverse at that point. Error ellipses provide an effective visual tool for inspecting positional accuracies of least squares adjusted station positions. All standard errors and error ellipse parameters in the latest version of EFBP are 95 % percent confidence values. The multiplier is a function of the redundancy (degrees of freedom) of your survey.

(4) What is the meaning of the "Band is xx stations" which appears just prior to the adjustment process itself?

The equations which are being solved tend to be "sparse". Sparse means there are lots of zero coefficients. If one can figure out where the zeros are, the station order can be switched until the zeros are ordered in some systematic fashion. One approach to re-ordering equations or stations is bandwidth optimization. It is used by many adjustment programs. The band is the "width" of the equations with the zero terms ordered outside of the non-zero band. A 1000 station network with a band of 20 stations will solve much faster than one with a band of 100 stations. The band is a function of redundancy and the number of traverse leg connections.
The total number of terms in a banded solution is a function of band size and the number of stations. Without algorithms which take advantage of sparsity we would be waiting a lot longer for solutions to occur.

13.6 Conclusions

Least squares adjustment is a statistically valid approach which can solve any survey network configuration simultaneously. With efficient software, a user can become proficient at using least squares in a short time. Hands-on experience with real data sets is the best learning tool that exists. One should not abuse the use of least squares adjustment. A user should always examine that amounts of adjustment to measurements are within acceptable limits.

Least squares adjustment does not eliminate the need for preservation of our measurement information. It does validate whether or not our measurements appear reliable. EFBP is an integrated data processing with least squares analysis. Hopefully this information will allow hands-on experience to be a more fruitful learning procedure.

13.7 Statistical Analyses in analyzing data using EFBP

EFBP includes a variety of statistical analyses which may initially be foreign to many land surveyors, but will become valuable tools with very minimal training.

Specific items which need to be considered in this discussion are:

(1) error estimation,
(2) standard error of unit weight/ chi-squared test,
(3) coordinate standard errors and error ellipses,
(4) validity of use of state plane coordinates in the adjustment process.

13.8 Basic Concepts of Error Estimation in Least Squares Analysis

While the majority of these concepts can be obtained in the understanding least squares analysis paper it doesn't hurt to say it again.

While many initial users are perplexed at the concept of error estimation it is really something one does in any analysis of survey data. One example of this is when one looks at a traverse closure and deems it acceptable or not. You are using your "apriori" knowledge of what errors exist in your measurements. Prior to least squares we actually prioritized our sections of traverse in some well-thought out order. The first traverse is totally superior to subsequent ones in this scenario, and the subsequent ones cannot affect the first traverse's coordinates. This is error estimation in an absolute sense - data set A is so superior to data set B that B cannot affect A. This has been used in control coordinate superiority to our measurements.

When estimating distance, angle, and azimuth error the key item to remember is "ESTIMATION". You are not required to absolutely define error in the way error propagation is defined in a textbook. Instead you are merely giving a logical value based on your knowledge of the type of the survey, instrumentation used, the terrain, etc.

The least squares analysis will indicate to you if your error estimates do not seem to fit the quality of the data. The standard error of unit weight and residuals will all help in this analysis.

If these indicators tell you something is fitting much better or worse than predicted, and you are confident the data has been input correctly, it is then you must decide whether to refine your error estimates.

The true indicator of the quality of your adjustment are your residuals, which simply tell you how much adjustment had to be applied to the input control coordinate, distance, angle, or azimuth.

Note that the residual is associated with the error estimate - you expect a distance with a 0.05 ft. error estimate to adjust more than one with a 0.01 ft. error estimate.

Residuals larger than three times their respective error estimate are generally considered as suspect. Statistics says a residual larger than three times its error estimate is suspect at a 95 % confidence limit. This means you are 95% confident it is a blunder.

All residuals should not necessarily be less than its error estimate. Statistics says only 66 % of them should be in a normal distribution. A significant number of residuals greater than its error estimate is an indication of slightly optimistic error estimates.

13.9 Standard error of Unit Weight / Chi-Squared Test

The standard error of unit weight is a unitless quantity (like residual / error estimate)that is an overall indicator of the quality of your error estimates. In EFBP it was generically said that if that number is between 0.7 and 2.5 you are doing a good job of error estimation. If you were perfect at error estimation the number would always be near one. The standard error of unit weight for perfect data would be zero but with crummy data it is open-ended on the large numerical side. This means the standard error of unit weight is not from a normal (bell-shaped) statistical distribution because it is not open-ended on the small end.

The standard error of unit weight can be statistically evaluated using a chi-squared test. This test is based on degrees of freedom (the number of measurements minus the number of unknown coordinates). The concept is that the more data you have the less variability there will be in your overall statistical indicators (outliers have less of an effect) and thus as your degrees of freedom goes up the tightness of your chi-squared test goes down. The chi-squared test thus is a low and high number range for the standard error of unit weight, and its range quits changing when the degrees of freedom exceeds 30. When your data exceeds 30 degrees of freedom your limits at a 95% confidence limit will always be 0.748 to 1.208 . This is also called the 5 % significance level. Meeting this range says your error estimates make sense at a 95% confidence level. If we were testing at the 99 % confidence level the range becomes larger since you want to be sure you are not messing with error estimates that are all right. This may seem backwards (higher confidence - bigger spread) but it has to do with the fact that in statistics you try to keep all data as is unless you have a real good reason to change it.

If you fail the test by a little it does not mean you should continue to edit error estimates as it will insignificantly change your final coordinates. This is why in EFBP the stated "limit" of the range was set arbitrarily higher in the documentation. You, as a surveyor, should evaluate if your residuals are too large, and act accordingly.

Why is it hard to pass the chi squared test? Are you 95% confident about anything that is related to measurements? Sometimes a question is the best answer.

13.10 Coordinate standard errors and error ellipses in EFBP

In all reality this is at best an uncertainty number because it gets bigger as things become more marginal.

The only real statistical way to define uncertainty about a position statistically is an error ellipse. An error ellipse is defined by two axis dimensions and the direction from north of the major axis.

In statistics error estimates of derived quantities such as coordinates are produced from t distributions which are a function of degrees of freedom and the desired confidence level. A low number of degrees of freedom results in a large multiplier to reach 95 % confidence as opposed to a large number of degrees of freedom. This makes sense since you cannot feel comfortable with your data until you have some redundancy. Like the chi-squared test, the t distribution multiplier quits changing after reaching 30 degrees of freedom where the multiplier of one sigma errors will be 2.51 .

The meaning of the produced coordinate error estimates is easy to explain using a cadastral survey scenario. You perform a dependent resurvey using all very permanent points, i.e., your traverse points aren't going to move. You run an adjustment of your data and produce coordinates and error estimates. If you went out and did the survey over under the same conditions, using the same instrumentation, the same techniques, and the same control, you would be 95 % confident that you could recompute all coordinates within the one you initially determined plus or minus its error estimate.

In a field survey sense you can thus think of it as how confident you would be at reproducibility of the coordinates.

Chapter 14. Geodesy and state plane coordinates and its relation to EFBP

14.1 Description of terms

Geodesy as it relates to EFBP is best explained by a series of elongated descriptions of some terms.

NAD 27 - North American Datum of 1927 - a datum based on the Clarke Ellipsoid of 1866, and fixed position and orientation at Meade's Ranch in Kansas. Based upon this information latitudes and longitudes were calculated for all National Geodetic Survey control points based primarily on triangulation. The calculation involved a least squares adjustment by hand calculations!

NAD 83 - North American Datum of 1983 - a datum based on the GRS 80 Ellipsoid, and fixed position and orientation at the center of the earth. The measurements used in NAD 27, and additional measurements made by the National Geodetic Survey subsequent to 1927, were subjected to a least squares adjustment - this time using a computer. This results in a different latitude and longitude for a station when compared to its NAD 27 values, with the difference often amounting to hundreds of feet.

State Plane Coordinates - projections which uniquely relates a geodetic position (latitude, longitude) to a plane coordinate pair (N,E). The relation is truly a one-to-one correspondence; for a unique state plane coordinate pair there is only one latitude, longitude pair, and vice-versa. The two types of projections that exist are Lambert Conformal - a conic projection for states elongated east-west, and Transverse Mercator - a cylindrical projection for states elongated north-south. The size of a zone was limited by defining a grid distance will not differ from its sea level counterpart by 1/10000 of the distance. Thus larger states are covered by more than one zone.

NAD 27 state plane coordinates - These are based on zone constants which include false northings and eastings so that negative coordinates do not result. The published units are U.S. Survey Feet.

NAD 83 state plane coordinates - These are based on different zone constants than NAD 27. The most notable change is that different values were assigned for false northings and eastings, and the coordinates published by NGS are in meters instead of U.S. Survey Feet. Even when converted to feet, NAD 83 coordinates look dramatically different than the NAD 27 coordinates for the same point because the false northings and eastings are different.

Geodetic distance - a horizontal distance between two points measured on the earth at the average elevation of the endpoints

Grid distance - a geodetic distance which has had scale and elevation factors applied to it to reduce it to the state plane zone.

Scale factor - Since a state plane zone intersects the earth geodetic distances must be projected to the zone. If the distance is inside where the zone intersects the earth the scale factor is less than one since the projection is inward. If the distance is outside where the zone intersects the earth the scale factor is greater than one since the projection is outward. The scale factor for a distance is the average of the scale factors at the two end points. The value of the scale factor is a function of location in the zone, which has a one-to-one relationship with geodetic position.

Elevation factor - Both the ellipsoids that represent NAD 27 and NAD 83 are near sea level, and thus any measured distance must be reduced to a sea level equivalent by the elevation factor. The elevation factor is based on the average of the elevations at the two end points of a line.

Elev. Factor = R / (R+h) where R = radius of the earth (20,906,000 ft)
h = elevation in feet

For metric reductions convert the radius to meters and use elevation in meters.

Relation of Geodetic and Grid Distance

Grid Distance = Geodetic Distance * Scale Factor * Elevation Factor

Geodetic Distance = Grid Distance / (Scale Factor * Elevation Factor)

Geodetic forward azimuth - the direction of a line relative to geodetic north

Geodetic back azimuth - the direction of the line relative to geodetic north in the opposite direction of the forward azimuth

Note: The forward and back geodetic azimuths do not differ by 180 degrees since north (meridian) lines are not parallel - they converge towards the north pole

Geodetic mean bearing (azimuth) - the average of the forward and the back azimuth. It is a line of constant bearing (a curved line on the earth). It is equivalent to the path you would follow if walking along a compass bearing. Section and township lines are defined in the Manual of Instructions to be mean bearing lines.

Grid north - the direction of a line relative to state plane grid north. It will not equal geodetic north since in the state plane grid all north directions are parallel and in a geodetic system north lines converge to the poles

Convergence angle - the angle between grid and geodetic north at a point. It is a function of location in the zone.

Second term correction - If converting a geodetic azimuth for a line to a grid azimuth (or vice-versa), a small correction must be applied to convergence angle which accounts for the fact that a geodetic azimuth exists on a curved surface (the earth) and a grid azimuth is on a flat surface (the projection). It is a function of the length of the line and its location in the zone, and is generally less than an arc second. This correction must also be applied to horizontal angles to reduce them to their grid equivalents. The second term correction must also be applied to horizontal angles to obtain the grid equivalent value, though the correction is often less than 0.1 seconds, so it is insignificant.

Relation of Grid and Geodetic Azimuth

Grid Azimuth = Geodetic Azimuth - Conv. Angle + second term correction

Geodetic Azimuth = Grid Azimuth + Conv. Angle - second term correction

14.2 Validity of use of state plane coordinates in the adjustment process

This issue has mainly been a constant source of criticism from certain people who do not understand how things can be performed geodetically sound in state plane coordinates if one applies correct scale, elevation, and convergence factors. This is quite an irony since when one works in assumed coordinates we are actually assuming a flat earth and that meridians do not converge toward the poles. State plane has often not been used because inversing coordinates in a plane mode gives grid (not ground) values. More COGO systems now allow you to obtain ground distances, ground acreages at an average elevation, and geodetic bearings from grid coordinates.

To test the validity of using state coordinates the author has been responsible for a variety of tests of "perfect" townships which are of course fictitious in nature. The data was generated using an external software system.

The data was tested at project datums, multiple elevations, and in the middle and slightly outside the limits of both Lambert and Mercator state plane zones. The least squares, which is operating in state plane, should show inaccuracies in the form of residuals if state plane coordinates could not be used. No problems were detected (all residuals were less than .001 ft. and .1 second) though it should still be determined how far outside a zone one has to go before problems occur.

Chapter 15. Electronic Field Book Processing Messages - How to troubleshoot problems in your data set

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15.1 Getting started

Before starting make sure the following files are in the directory that contains your data:

Required files:
DEFAULT.SD
DEFAULT.CON

The DEFAULT files are used for error estimate definition in the least squares processing and other various items. If the DEFAULT files do not exist reasonable error estimates will be automatically defined.
In any case when you run EFBP you will see a menu run allows you to change the project or the variety of items in the DEFAULT files.

15.2 Understanding error messages

Understanding error messages

The key to success in processing data through EFBP is correctly interpreting the messages which come from processing. It is important to note that errors in the actual readings from the total station are uncommon.

The blunders that occur are due to human input or misinterpretation.
They include:

(1) station misnaming - shooting A23 but calling it A22
(2) missing setup - EFB does not know when you have moved the instrument.
You tell it by storing a setup record.
(3) accidently turning the lower motion
(4) accidently zeroing the horizontal circle
(5) accidently toggling the position number
(6) forgetting to toggle the position number
(7) incorrect height of instrument
(8) incorrect height of target
(9) incorrect control station reference name
(10) incorrect control coordinates

The error message may be in the form of a large repetition error or an ignored set of data. This information will help you debug your blunders.

15.3 EFBP Error Estimation Generation Messages

COMPUTED STANDARD ERRORS WILL BE USED

Processing has two options for generating error estimates for the least squares analysis. If one selects to use computed error estimates the standard error of the mean is used plus a user selected "add-on" which accounts for the fact that repetition error does not model random errors such as error in setup. If a station is not shot multiple times the error estimates defined by the user (not add ons) are used. If a user chooses not to use error estimates computed from repetition the error estimates defined by the user are used for all measurements.

15.4 EFBP Control Input Messages

CONTROL INFORMATION FOUND WITHOUT STATION NAME

Coordinates exist in the control database with a blank station name.

COORDINATE INFO. NOT FOUND AFTER CONTROL STATION NAME

A station name exists in the control database without any coordinates.

STATION A23 IN CONTROL FILE TWICE

This should never happen.

15.5 EFBP Abstracting Messages

When required the data in an .obs file will be shown to help illustrate the problem and the solution. The .obs file is the ASCII version of a raw data file (.RAW). The EFB data collection and processing manuals should be used to understand the format of a .OBS file. Abstracting means the meaning of measurements which have been repeated.

FOLLOWING OBSERVATIONS DETECTED W/O SETUP - DELETED

Measurements existed which are not attached to a setup record. This should only be possible at the start of a measurement file.

PROCESSING SETUP # 1 AT STATION SP1

REPETITION STANDARD ERRORS

SIGHTED	HORIZONTAL DISTANCE			ELEVATION DIFF.			COMPARE	
STATION	SD	SD (MEAN)	MAX SPREAD	SD	SD (MEAN)	MAX SPREAD	HORZ. DIST.	ELEV. DIFF.
BM1	.006	.003	.005	.006	.003	.008		
FCI1	.001	.001	.001	.089	.063	.063		
NP1	.005	.002	.007	.021	.010	.022	.007	-.047
TOM1	.005	.003	.008	.009	.005	.010		

REPETITION STANDARD ERRORS FOR ANGLES

BS STATION   FS STATION   SD   SD (MEAN)   MAX SPREAD

```
 BM1            FCI1           35.       25.          25.
 BM1            NP1            19.        9.          25.
 BM1            TOM1           16.        8.          20.
```

This indicates your pointing error if no station naming problems exist. If some of these are large it probably indicates a station naming problem, a position number problem, or a missing setup. The maximum spread is the largest deviation of a single shot from the mean. The "compare horz. dist. and elev. diff." is shown if that line has been shot previously from a different setup record. It may be at the same setup or a setup at the sighted station. If all repetition errors are small but the "compare horizontal distance" is large you should look for a station naming problem at this setup or a previous setup which measured that line. A good compare horz. dist. but a poor elev. diff. indicates a height of instrument (HI) or height of target (HT) at this setup or a previous setup on which the line was measured. If all angle errors are large it is a backsight station problem (station naming or position #). If only one station has a large angle error problem concentrate on the same problems only on the shots to that station. All distance and elevation error values are in the same units used in the data. All angle repetition error information is in seconds.

These repetition results were derived from the following data:

```
S 00 SP1                        P G1
S 01 16:21:59 11/23/88  3.680
O 00 BM1      55-84-A09         P G1
O 01 16:32:52    5.000            1 R 354 45 13.0 269 59 39     184.640 S
O 01 16:33:53    5.000            1 D 174 45 46.0  90  0 28     184.640 S
O 01 16:42:43    5.000            2 R  88 24 41.0 269 59 46     184.630 S
O 01 16:41:28    5.000            2 D 268 24 17.0  90  0 27     184.630 S
O 00 FCI1     55-72-A16         P G1
O 01 16:24:15    5.850            1 R   0  0  1.0 270 53 49    3692.450 S
O 01 16:38:03    5.850            1 D 179 59 44.0  89  6  4    3692.450 S
O 00 NP1      55-84-19A         P G1
O 01 16:35:39    5.140            1 D 359 49 31.0  90 32 39     511.230 S
O 01 16:25:39    5.140            1 R 179 49 24.0 269 27 38     511.220 S
O 01 16:44:59    5.140            2 R 273 28 55.0 269 27 36     511.230 S
O 01 16:40:04    5.140            2 D  93 28 48.0  90 32 35     511.230 S
O 00 TOM1                       P G1
O 01 16:34:51    5.360            1 D  38 19  6.0  93 37  5     505.650 S
O 01 16:28:43    5.360            1 R 218 18 59.0 266 23  0     505.650 S
O 01 16:40:40    5.360            2 D 131 58 14.0  93 37  6     505.640 S
O 01 16:43:37    5.360            2 R 311 58 16.0 266 23  2     505.650 S
```

The .obs file is very easy to read. The first two lines represent a setup at SP1 with a H.I. of 3.680 . Four measurements were made to BM1 in HVD mode. This mode is indicated by the 01 record in columns 3-4. Some other examples of mode are 02 represents HD (horizontal circle, horizontal distance), 04 represents H (horizontal circle only, and 09 represents level rod reading(s). The first measurement to BM1 was made in position #1 reverse with horizontal circle, zenith circle, and slope distance of 354 45 13.0, 269 59 39, and 184.640 respectively. The third and fourth measurements to BM1 were made in position #2.

The collimation errors were assumed to be zero for this example. Since EFB processing allows each shot to be corrected for systematic error derived from horizontal and vertical collimation, each shot is treated individually. 1 direct and 1 reverse has been conventionally averaged, then repetition error computed since collimation error has not been measured - in contrast to this 4 direct and 4 reverse shots are treated as 8 measurements in computing standard errors in EFB processing since collimation errors can be corrected using calibration.

The compare horizontal distance and elevation difference was derived from a setup at NP1 from which the following shots were made to SP1:

```
S 00 NP1      55-84-19A        P G1
S 01 14:18:00 11/23/88  5.400
O 00 SP1                       P G1
O 01 14:26:55    3.430           1 D 359 50 52.0  89 31 30     511.230 S
O 01 14:36:18    3.430           1 R 179 50 36.0 270 28 55     511.230 S
O 01 14:51:38    3.430           4 D 162 41 51.0  89 31 18     511.230 S
O 01 14:57:58    3.430           4 R 342 41 42.0 270 28 57     511.230 S
O 01 15:01:30    3.430           5 R 250 45 28.0 270 28 49     511.230 S
O 01 15:08:29    3.430           5 D  70 45 34.0  89 31 21     511.230 S
```

Now consider in the example setup at SP1 that TOM1 and NP1 were misnamed for each other as shown:

```
S 00 SP1                       P G1
S 01 16:21:59 11/23/88  3.680
O 00 BM1      55-84-A09        P G1
O 01 16:32:52    5.000           1 R 354 45 13.0 269 59 39     184.640 S
O 01 16:33:53    5.000           1 D 174 45 46.0  90  0 28     184.640 S
O 01 16:42:43    5.000           2 R  88 24 41.0 269 59 46     184.630 S
O 01 16:41:28    5.000           2 D 268 24 17.0  90  0 27     184.630 S
O 00 FCI1     55-72-A16        P G1
O 01 16:24:15    5.850           1 R   0  0  1.0 270 53 49    3692.450 S
O 01 16:38:03    5.850           1 D 179 59 44.0  89  6  4    3692.450 S
O 00 TOM1                      P G1
O 01 16:35:39    5.140           1 D 359 49 31.0  90 32 39     511.230 S
O 01 16:25:39    5.140           1 R 179 49 24.0 269 27 38     511.220 S
O 01 16:44:59    5.140           2 R 273 28 55.0 269 27 36     511.230 S
O 01 16:40:04    5.140           2 D  93 28 48.0  90 32 35     511.230 S
O 00 NP1      55-84-19A        P G1
O 01 16:34:51    5.360           1 D  38 19  6.0  93 37  5     505.650 S
O 01 16:28:43    5.360           1 R 218 18 59.0 266 23  0     505.650 S
O 01 16:40:40    5.360           2 D 131 58 14.0  93 37  6     505.640 S
O 01 16:43:37    5.360           2 R 311 58 16.0 266 23  2     505.650 S
```

The repetition errors would be reported as:

```
 PROCESSING SETUP #   1 AT STATION SP1
 REPETITION STANDARD ERRORS
SIGHTED      HORIZONTAL DISTANCE      ELEVATION DIFF.           COMPARE
STATION      SD      SD     MAX        SD     SD       MAX      HORZ.     ELEV.
                  (MEAN)  SPREAD          (MEAN)    SPREAD      DIST.     DIFF.
BM1         .006    .003    .005      .006   .003      .008
FCI1        .001    .001    .001      .089   .063      .063
TOM1        .005    .002    .007      .021   .010      .022
NP1         .005    .003    .008      .009   .005      .010  *  6.572* *-27.336*
 REPETITION STANDARD ERRORS FOR ANGLES
 BS STATION     FS STATION     SD    SD (MEAN)  MAX SPREAD
 BM1            FCI1           35.       25.         25.
 BM1            TOM1           19.        9.         25.
 BM1            NP1            16.        8.         20.
```

Notice the repetition errors are fine as the station misnaming was very consistent. The compare horizontal distance and elev. diff. caught the problem. Beware, when comparing two horizontal distances of the same line, that the second measurement, not the first, may be the correct one!

Asterisks (*) appear around max. spreads or compares when they are above user defined values.

The next example is where station names of NP1 and TOM1 were only reversed on the first position in reverse:

```
S 00 SP1                        P G1
S 01 16:21:59 11/23/88  3.680
O 00 BM1      55-84-A09         P G1
O 01 16:32:52    5.000            1 R 354 45 13.0 269 59 39     184.640 S
O 01 16:33:53    5.000            1 D 174 45 46.0  90  0 28     184.640 S
O 01 16:42:43    5.000            2 R  88 24 41.0 269 59 46     184.630 S
O 01 16:41:28    5.000            2 D 268 24 17.0  90  0 27     184.630 S
O 00 FCI1     55-72-A16         P G1
O 01 16:24:15    5.850            1 R   0  0  1.0 270 53 49    3692.450 S
O 01 16:38:03    5.850            1 D 179 59 44.0  89  6  4    3692.450 S
O 00 NP1      55-84-19A         P G1
O 01 16:34:51    5.360            1 D  38 19  6.0  93 37  5     505.650 S
O 01 16:25:39    5.140            1 R 179 49 24.0 269 27 38     511.220 S
O 01 16:44:59    5.140            2 R 273 28 55.0 269 27 36     511.230 S
O 01 16:40:04    5.140            2 D  93 28 48.0  90 32 35     511.230 S
O 00 TOM1                       P G1
O 01 16:35:39    5.140            1 D 359 49 31.0  90 32 39     511.230 S
O 01 16:28:43    5.360            1 R 218 18 59.0 266 23  0     505.650 S
O 01 16:40:40    5.360            2 D 131 58 14.0  93 37  6     505.640 S
O 01 16:43:37    5.360            2 R 311 58 16.0 266 23  2     505.650 S
```

```
 PROCESSING SETUP #   1 AT STATION SP1
 REPETITION STANDARD ERRORS
SIGHTED      HORIZONTAL DISTANCE      ELEVATION DIFF.           COMPARE
STATION      SD      SD     MAX        SD     SD       MAX      HORZ.     ELEV.
                  (MEAN)  SPREAD          (MEAN)    SPREAD      DIST.     DIFF.
BM1         .006    .003    .005      .006   .003      .008
FCI1        .001    .001    .001      .089   .063      .063
NP1        3.281   1.640 * 4.921*  13.652  6.826 * 20.478* *  1.649* * -6.865*
```

```
TOM1      3.284   1.642 * 4.926*  13.632  6.816 * 20.448*
 REPETITION STANDARD ERRORS FOR ANGLES
 BS STATION      FS STATION     SD    SD (MEAN)  MAX SPREAD
 BM1             FCI1           35.       25.         25.
 BM1             NP1         69271.    34635.     ********
 BM1             TOM1        69301.    34650.     ********
```

Note how the error appears consistent between the two stations. The ***'s under max spread for angles is not for good work, the value does not fit in the reserved five places to the right of the decimal! It is very important to remember similar "poor" repetition errors to two stations often indicates station misnaming between the two.

A four minute pointing blunder is now placed in position #2 reverse of BM1:

```
S 00 SP1                        P G1
S 01 16:21:59 11/23/88  3.680
O 00 BM1      55-84-A09         P G1
O 01 16:32:52    5.000            1 R 354 45 13.0 269 59 39     184.640 S
O 01 16:33:53    5.000            1 D 174 45 46.0  90  0 28     184.640 S
O 01 16:42:43    5.000            2 R  88 24 41.0 269 59 46     184.630 S
O 01 16:41:28    5.000            2 D 268 28 17.0  90  0 27     184.630 S
O 00 FCI1     55-72-A16         P G1
O 01 16:24:15    5.850            1 R   0  0  1.0 270 53 49    3692.450 S
O 01 16:38:03    5.850            1 D 179 59 44.0  89  6  4    3692.450 S
O 00 NP1      55-84-19A         P G1
O 01 16:35:39    5.140            1 D 359 49 31.0  90 32 39     511.230 S
O 01 16:25:39    5.140            1 R 179 49 24.0 269 27 38     511.220 S
O 01 16:44:59    5.140            2 R 273 28 55.0 269 27 36     511.230 S
O 01 16:40:04    5.140            2 D  93 28 48.0  90 32 35     511.230 S
O 00 TOM1                       P G1
O 01 16:34:51    5.360            1 D  38 19  6.0  93 37  5     505.650 S
O 01 16:28:43    5.360            1 R 218 18 59.0 266 23  0     505.650 S
O 01 16:40:40    5.360            2 D 131 58 14.0  93 37  6     505.640 S
O 01 16:43:37    5.360            2 R 311 58 16.0 266 23  2     505.650 S
```

This results in repetition errors of:

```
 PROCESSING SETUP #   1 AT STATION SP1
 REPETITION STANDARD ERRORS
SIGHTED      HORIZONTAL DISTANCE      ELEVATION DIFF.         COMPARE
STATION      SD     SD     MAX        SD     SD      MAX      HORZ.     ELEV.
              (MEAN)  SPREAD              (MEAN)   SPREAD    DIST.     DIFF.
BM1        .006   .003    .005      .006   .003     .008
FCI1       .001   .001    .001      .089   .063     .063
NP1        .005   .002    .007      .021   .010     .022      .007     -.047
TOM1       .005   .003    .008      .009   .005     .010
 REPETITION STANDARD ERRORS FOR ANGLES
 BS STATION      FS STATION     SD    SD (MEAN)  MAX SPREAD
 BM1             FCI1           35.       25.         25.
 BM1             NP1           107.       53.     *  159.*
 BM1             TOM1          109.       54.     *  162.*
```

Note the four minutes gets slightly masked by averaging. The shots to FC1 were unaffected since it was not shot on position #2 reverse. The repetition errors are consistent to both NP1 and TOM1 indicating an error in the backsight station.

Now lets place the four minute blunder in TOM1's position #2 reverse:

```
S 00 SP1                        P G1
S 01 16:21:59 11/23/88  3.680
O 00 BM1      55-84-A09         P G1
O 01 16:32:52    5.000            1 R 354 45 13.0 269 59 39     184.640 S
O 01 16:33:53    5.000            1 D 174 45 46.0  90  0 28     184.640 S
O 01 16:42:43    5.000            2 R  88 24 41.0 269 59 46     184.630 S
O 01 16:41:28    5.000            2 D 268 24 17.0  90  0 27     184.630 S
O 00 FCI1     55-72-A16         P G1
O 01 16:24:15    5.850            1 R   0  0  1.0 270 53 49    3692.450 S
O 01 16:38:03    5.850            1 D 179 59 44.0  89  6  4    3692.450 S
O 00 NP1      55-84-19A         P G1
O 01 16:35:39    5.140            1 D 359 49 31.0  90 32 39     511.230 S
O 01 16:25:39    5.140            1 R 179 49 24.0 269 27 38     511.220 S
O 01 16:44:59    5.140            2 R 273 28 55.0 269 27 36     511.230 S
O 01 16:40:04    5.140            2 D  93 28 48.0  90 32 35     511.230 S
O 00 TOM1                       P G1
O 01 16:34:51    5.360            1 D  38 19  6.0  93 37  5     505.650 S
O 01 16:28:43    5.360            1 R 218 18 59.0 266 23  0     505.650 S
O 01 16:40:40    5.360            2 D 131 58 14.0  93 37  6     505.640 S
O 01 16:43:37    5.360            2 R 311 54 16.0 266 23  2     505.650 S
```

This results in a repetition report of:

```
 PROCESSING SETUP #   1 AT STATION SP1
 REPETITION STANDARD ERRORS
SIGHTED      HORIZONTAL DISTANCE      ELEVATION DIFF.          COMPARE
STATION      SD      SD      MAX       SD     SD      MAX     HORZ.     ELEV.
                   (MEAN)  SPREAD            (MEAN)  SPREAD   DIST.     DIFF.
BM1        .006    .003    .005      .006   .003     .008
FCI1       .001    .001    .001      .089   .063     .063
NP1        .005    .002    .007      .021   .010     .022      .007     -.047
TOM1       .005    .003    .008      .009   .005     .010
 REPETITION STANDARD ERRORS FOR ANGLES
 BS STATION      FS STATION      SD    SD (MEAN)  MAX SPREAD
 BM1             FCI1            35.       25.         25.
 BM1             NP1             19.        9.         25.
 BM1             TOM1           124.       62.     *  185.*
```

Note the 4 minute blunder is again slightly masked but it is concentrated on the angle to TOM1.

ERROR WHEN BACKSIGHT RESHOT IS 0.004

In differential leveling if the backsight is reshot this message provides one with the difference between the initial and final readings.

```
REPETITION ERROR ON MULTIPLE POINTING TO STATION FCI1   IS  8.0  SEC.
```

This is standard when reshooting the backsight after a lot of foresights. If you did not do this and this message appears you have a station naming problem or a missing setup. You have measured to the same station more than once with the same position number and in the same D/R mode. If this is very large you have a station naming problem, a position # problem, a missing setup, you used the lower motion inadvertently, or you zeroed the instrument inadvertently.

In the following .obs information the 1st position reverse value of FCI1 was incorrectly called BM1. You can usually find this problem by observing two similar position numbers and D/R readings and noting different circle readings and slope distances.

```
S 00 SP1                          P G1
S 01 16:21:59 11/23/88  3.680
O 00 BM1      55-84-A09           P G1
O 01 16:24:15    5.850              1 R   0  0  1.0 270 53 49    3692.450 S
O 01 16:32:52    5.000              1 R 354 45 13.0 269 59 39     184.640 S
O 01 16:33:53    5.000              1 D 174 45 46.0  90  0 28     184.640 S
O 01 16:42:43    5.000              2 R  88 24 41.0 269 59 46     184.630 S
O 01 16:41:28    5.000              2 D 268 24 17.0  90  0 27     184.630 S
O 00 FCI1     55-72-A16           P G1
O 01 16:38:03    5.850              1 D 179 59 44.0  89  6  4    3692.450 S
O 00 NP1      55-84-19A           P G1
O 01 16:35:39    5.140              1 D 359 49 31.0  90 32 39     511.230 S
O 01 16:25:39    5.140              1 R 179 49 24.0 269 27 38     511.220 S
O 01 16:44:59    5.140              2 R 273 28 55.0 269 27 36     511.230 S
O 01 16:40:04    5.140              2 D  93 28 48.0  90 32 35     511.230 S
O 00 TOM1                         P G1
O 01 16:34:51    5.360              1 D  38 19  6.0  93 37  5     505.650 S
O 01 16:28:43    5.360              1 R 218 18 59.0 266 23  0     505.650 S
O 01 16:40:40    5.360              2 D 131 58 14.0  93 37  6     505.640 S
O 01 16:43:37    5.360              2 R 311 58 16.0 266 23  2     505.650 S
```

The resultant processing information looks like this:

```
 PROCESSING SETUP #   1 AT STATION SP1
 REPETITION ERROR ON MULTIPLE POINTING TO STATION BM1 IS * 18888.0* SEC.
 REPETITION STANDARD ERRORS
SIGHTED      HORIZONTAL DISTANCE      ELEVATION DIFF.          COMPARE
STATION      SD      SD     MAX        SD     SD      MAX     HORZ.     ELEV.
                   (MEAN)  SPREAD            (MEAN)  SPREAD   DIST.     DIFF.
BM1     ******* 701.473 ********  25.479 11.394 * 45.577*
NP1        .005    .002    .007     .021   .010     .022      .007     -.047
TOM1       .005    .003    .008     .009   .005     .010
 REPETITION STANDARD ERRORS FOR ANGLES
 BS STATION     FS STATION     SD    SD (MEAN)  MAX SPREAD
 BM1            NP1          4722.     2361.     * 7082.*
 BM1            TOM1         4718.     2359.     * 7076.*
```

It is obvious that some data shot to BM1 is incorrect. The easiest process for finding this kind of problem is realizing that if the instrument is not horizontally re-zeroed the reverse horizontal circle should be 180 degrees different from the direct horizontal circle reading. If you knew no stations had been reshot on the same position number with the same D/R reading one would immediately realize a station naming problem had occurred.

NOT ALL ANGLES HAVE A COMMON BACKSIGHT

This means stations on a position # and unique D/R are not connected to any other stations on other position #s and unique D/R. This is not allowed in processing. If you really intended to do this it must be a separate setup. In the following data the last shot by time tag was to NP1 and the position number was accidentally changed from 2 to 3.

```
S 00 SP1                          P G1
S 01 16:21:59 11/23/88  3.680
O 00 BM1      55-84-A09           P G1
O 01 16:32:52    5.000              1 R 354 45 13.0 269 59 39     184.640 S
O 01 16:33:53    5.000              1 D 174 45 46.0  90  0 28     184.640 S
O 01 16:42:43    5.000              2 R  88 24 41.0 269 59 46     184.630 S
O 01 16:41:28    5.000              2 D 268 24 17.0  90  0 27     184.630 S
O 00 FCI1     55-72-A16           P G1
O 01 16:24:15    5.850              1 R   0  0  1.0 270 53 49    3692.450 S
O 01 16:38:03    5.850              1 D 179 59 44.0  89  6  4    3692.450 S
O 00 NP1      55-84-19A           P G1
O 01 16:35:39    5.140              1 D 359 49 31.0  90 32 39     511.230 S
O 01 16:25:39    5.140              1 R 179 49 24.0 269 27 38     511.220 S
O 01 16:44:59    5.140              3 R 273 28 55.0 269 27 36     511.230 S
O 01 16:40:04    5.140              2 D  93 28 48.0  90 32 35     511.230 S
O 00 TOM1                         P G1
O 01 16:34:51    5.360              1 D  38 19  6.0  93 37  5     505.650 S
O 01 16:28:43    5.360              1 R 218 18 59.0 266 23  0     505.650 S
O 01 16:40:40    5.360              2 D 131 58 14.0  93 37  6     505.640 S
O 01 16:43:37    5.360              2 R 311 58 16.0 266 23  2     505.650 S
```

It resulted in the following processing information.

```
 PROCESSING SETUP #   1 AT STATION SP1
 REPETITION STANDARD ERRORS
SIGHTED      HORIZONTAL DISTANCE      ELEVATION DIFF.          COMPARE
STATION      SD     SD     MAX          SD     SD      MAX      HORZ.     ELEV.
                  (MEAN)  SPREAD              (MEAN)  SPREAD    DIST.     DIFF.
BM1        .006    .003    .005       .006   .003     .008
FCI1       .001    .001    .001       .089   .063     .063
NP1        .005    .002    .007       .021   .010     .022      .007     -.047
TOM1       .005    .003    .008       .009   .005     .010
 REPETITION STANDARD ERRORS FOR ANGLES
 BS STATION     FS STATION     SD    SD (MEAN)  MAX SPREAD
 NOT ALL ANGLES HAVE A COMMON BACKSIGHT
 BM1            FCI1           35.       25.         25.
 BM1            NP1            23.       13.         24.
 BM1            TOM1           16.        8.         20.
```

```
SOME DATA AT STATION SP1      NOT CONNECTED TO OTHER
POINTINGS SO IGNORED
```

The processed data would simply not use the horizontal circle reading which was incorrectly recorded as the 3rd position. The distance and elevation information would be used.

```
SOME DATA AT STATION A23 NOT CONNECTED TO OTHER
POINTINGS SO IGNORED
```

This means stations on a position # and unique D/R are not connected to any other stations on other position #s and unique D/R. This is not allowed in processing. If you really intended to do this it must be a separate setup. There are two ways EFBP searches for this problem and thus the two error messages exist for what appears to be the same problem. Notice this message also appeared in the previous example.

```
IGNORED CALIBRATION - # OF DIRECT READINGS DOES NOT
EQUAL NUMBER OF REVERSE READINGS
```

If the D/R pointings are not equal in number the calibration is ignored and the previous calibration values are retained.

```
WARNING - CALIBRATION RECORD WITHOUT DATA
```

A calibration record is stored even when you do not perform the vertical and horizontal collimation checking process. If you did not intend to perform this test ignore this warning. The previous calibration values are retained.

```
HORZ. COLLIMATION CORRECTION =    3.6 SECONDS
VERT. COLLIMATION CORRECTION =  -18.3 SECONDS
```

These should remain fairly consistent for a particular instrument. If they do not something in the instrument or setup is very unstable. If these are unusually large you did not perform this test properly or instrument is very out of adjustment. A very different calibration correction compared to others should probably be deleted as something was wrong regarding the instrument (stuck collimator) or the setup.

```
HORIZONTAL POINTING STANDARD ERROR  (DIRECT) = 3.2 SECONDS
VERTICAL POINTING STANDARD ERROR  (DIRECT) =  5.1 SECONDS
HORIZONTAL POINTING STANDARD ERROR (REVERSE) = 1.1 SECONDS
VERTICAL POINTING STANDARD ERROR (REVERSE) = 7.8 SECONDS
```

If these are large you did a poor job of pointing. Horizontally you may have inadvertently zeroed the instrument or used the lower motion. Delete the calibration if unusually large pointing errors exist. An example of a poor calibration is:

```
C 00 11:49:48 09/06/90  80 30.0 00120  DWH HEC RLB
C 01 TOPCON     GTS-4      W60092      5  3
C 03 12:39:26                      D 274 58  9.0  86 44 24.0
C 03 12:39:50                      D 274 58 11.0  86 44 26.0
C 03 12:39:26                      D 274 58  8.0  86 43 11.0
C 03 12:39:50                      D 274 58  7.0  86 44 21.0
C 03 12:42:00                      R  94 58 32.0 273 15 15.0
C 03 12:42:00                      R  94 58 36.0 273 15 10.0
C 03 12:42:00                      R  94 58 38.0 273 15 12.0
C 03 12:42:18                      R  94 58 30.0 273 15 13.0
```

The processing of it results in:

```
 HORZ. COLLIMATION CORRECTION =     12.6 SECONDS
 VERT. COLLIMATION CORRECTION =     21.0 SECONDS
 HORIZONTAL POINTING STANDARD ERROR  (DIRECT) =      1.7 SECONDS
   VERTICAL POINTING STANDARD ERROR  (DIRECT) =     36.4 SECONDS
 HORIZONTAL POINTING STANDARD ERROR (REVERSE) =      3.7  SECONDS
   VERTICAL POINTING STANDARD ERROR (REVERSE) =      2.1 SECONDS
```

While 36.4 seconds may not be considered large it is many magnitudes larger than the other pointing errors. Examining the data it is obvious the third pointing in direct was performed improperly vertically. To delete it you must also delete one reverse pointing to obtain equal number of D/R pointings.

15.6 EFBP Sideshot Identification Messages

----

112 OF 122 STATIONS ARE HORIZONTAL SIDESHOTS

EFB processing does not require a user to define what is a sideshot and what is part of the traverse data. Processing resolves this automatically. Only traverse (redundant) data is analyzed by least squares. Sideshots are computed after the least squares analysis is completed.

ERROR - ANGLE AT A SETUP AT STATION A23
NOT CONNECTED TO ANY SIGHTED STATIONS AT PREVIOUS SETUPS
PROCESSING CONTINUES BUT THIS PROBLEM NEEDS TO BE CHECKED

An example of this is an angle of Bs B22 Occ A23 Fs B23 where none of the 3 stations are control points and B22 and B23 have never been occupied or shot from other stations. There is little chance that coordinates can be solved in this situation and thus it is probably a station misnaming situation.

CANNOT REALIGN ANGLES AT STATION A23 DUE TO LACK OF ANGLE INFORMATION
PROCESSING CONTINUES BUT THIS PROBLEM NEEDS TO BE CHECKED

This occurs during sideshot elimination. If a sideshot was originally selected by processing as a backsight it must be changed to a foresight and a different backsight station selected (REALIGN). If during this process a suitable backsight which is common to all foresights cannot be found this warning is displayed. The most common occurrence of this message is when a setup on A23 occurs multiple times and one unique backsight cannot be found common to all setups. While this may be computationally solvable processing does not allow it. The solution is to use the same backsight on all setups of the same station. If this is not possible on the new setup shoot a previous foresight and processing can resolve this situation.

15.7 EFBP Coordinate Generation Messages

NO AZIMUTH FOUND AND ONLY ONE CONTROL POINT
COORDINATE GENERATION IS INCOMPLETE

The horizontal data ties to only one control point and thus without an azimuth coordinates cannot be generated. You may have forgot to enter the azimuth or a second control station. Incorrect station naming may also create this problem as the survey network has been misidentified.

NO DISTANCE TO CONTROL POINT. STOP

Only in a triangulation (bearing-bearing intersection) can you produce coordinates if no distances are connected to control stations.

NETWORK CONNECTIVITY IS INCOMPLETE

Station naming problems or missing setups have created a situation where the survey data does not permit coordinate generation. This is referred to as a lack of connectivity. An example would be a missing setup in a traverse between two control points.

ROTATION NOT POSSIBLE - MISSING DATA

Coordinate generation did not succeed in connecting two control points, or an azimuth is not on the data which has unrotated coordinates. Data is missing or station misnaming has caused this problem.

*** COORDINATES UNDETERMINED FOR STATION A23

If station A23 is part of your horizontal survey data this is a problem due to missing data or station misnaming. If station A23 is part of a differential leveling survey this message can be ignored as you did not intend to have coordinates generated for it.

STATION A23 IS NOT ON A DISTANCE IN INPUT FILE

If station A23 is part of your horizontal survey data this may be a problem due to missing data or station misnaming. If station A23 is part of a differential leveling survey this message can be ignored as you did not intend to have coordinates generated for it. If station A23 was intended to be resolved by bearing-bearing intersection or resection this message can again be ignored.

STATION A23 IS NOT ON AN ANGLE IN INPUT FILE

If station A23 is part of your horizontal survey data this may be a problem due to missing data or station misnaming. If station A23 is part of a differential leveling survey this message can be ignored as you did not intend to have coordinates generated for it. If station A23 which was intended to be resolved by distance-distance intersection this message can again be ignored.

NO COORDINATES FOR STATION A23 WRITTEN TO FILE

If station A23 is part of your horizontal survey data this is a problem due to missing data or station misnaming. If station A23 is part of a differential leveling survey this message can be ignored as you did not intend to have coordinates generated for it.

```
STATION  A4        POSITIONED BY ANGLE-ANGLE
INTERSECTION FROM STATIONS A1        AND A2
```

Unless this was intentional you may have missing distances.

```
STATION A5        POSITIONED BY RESECTION FROM
STATIONS A3A1      , A2       AND A1A2
```

Unless this was intentional you may have missing distances.

```
STATION A66       POSITIONED BY DISTANCE-DISTANCE INTERSECTION
FROM STATIONS A3        AND A2
```

Unless this was intentional you may have missing angles. This required user input to resolve a multiple solution.

```
STATION A6        POSITIONED BY THREE DISTANCE INTERSECTION
FROM STATIONS A1       , A3        , AND A2
```

Unless this was intentional you may have missing angles. If three distances exist the multiple solution can be automatically resolved.

```
STATION A18       POSITIONED BY ANGLE-DISTANCE INTERSECTION
FROM STATIONS A3        AND A4
```

Unless this was intentional you may have missing angles or distances.

This possibly required user input to resolve a multiple solution.

TRAVERSE CLOSURE REPORT
LINEAR ERROR OF CLOSURE IS .033 FT.

PRECISION IS 1/ 50739.

STATION	X COOR.	Y COOR.
B6	623323.994	158173.434
B25	623420.673	157621.313
B5	623433.784	157474.263
B1	623547.043	157322.610
B2	623668.589	157386.943
B24	623777.710	157511.384
B23	623904.038	157974.433

Once sideshots have been removed the processor begins coordinate generation by attempting to traverse between control points or points with previously computed coordinates. The processor does not compute traverse loops - only traverse links between existing coordinates. This is because the end point coordinates are necessary to define the proper rotation of the traverse link. A loop traverse requires a known bearing for rotation and this is not searched for by the processor. A compass rule adjustment of the link traverse is performed. If all abstracting messages show no problems, one should look at the traverse closures in sequential order. The first closure which illustrates a blunder indicates highly that a problem with that traverse link's data. This could be due to incorrect control coordinates or incorrect station names, and is probably not due to the values of the measurements as they have come directly from the total station in most cases.

Poor closures after the first bad closure should usually be ignored until the problem with the first traverse link is resolved. This is because later traverse links may start or end on coordinates generated from the first "bad" traverse. As an example if the above traverse closed 1/80 and a later computed traverse started at B25 and ended at B24 it would close poorly due to the poor coordinates of B25 and B24, and not necessarily due to the measurements of the later traverse link.

Closures tend to get slightly worse as later links are identified as you are closing between coordinates generated from series of compass rule adjustments. This problem is resolved by the least squares analysis.

If processing shows an angle-distance computation when you were expecting a traverse link closure computation make sure a missing setup or similar problem does not exist on that link traverse. Please be aware that an angle-distance computation is often necessary to get the link traverse closure computation functioning.

If a sideshot is measured to more than once and error estimates from repetition is selected that data will be in the redundant (traverse) data and not in the sideshot information. Since this station is not on a link traverse it will require an angle distance combination.

DISTANCE RESIDUALS

DISTANCE		RESIDUAL
A25	- B22	.000
A25	- B23	.000
B23	- B24	-.026
B23	- B6	.000
B24	- B2	-.009
B2	- B1	-.007
B1	- B5	-.010
B5	- B25	-.008
B25	- B6	-.030
GRASS2	- B2	-.078
GRASS2	- B1	-.160
GRASS2	- B5	-.088
GRASS2	- C1	.000
GRASS2	- XS1	.000

ANGLE RESIDUALS

ANGLE			RESIDUAL (SEC)
B22	- A25	- B23	.0
A25	- B23	- B24	-1.8
A25	- B23	- B6	.0
B2	- B24	- B23	.0
B1	- B2	- B24	.0
B2	- B1	- B5	.0
B1	- B5	- B25	.0
B5	- B25	- B6	.0
B23	- B6	- B25	-13.4
B2	- GRASS2	- B1	190.6
B2	- GRASS2	- B5	.0
B2	- GRASS2	- C1	.0
B2	- GRASS2	- XS1	.0

AZIMUTH RESIDUALS

AZIMUTH		RESIDUAL (SEC)
A25	- B22	.0

These residuals show the amounts of adjustment to your data based on the initial compass rule generation of coordinates. Data which has not been used to generate coordinates will contain the largest adjustments and you can sometimes detect a blunder if all of the traverse closures are good but a large residual is found in a measurement that was not used in coordinate generation. Be a little careful in over-interpretation as the least squares will provide you with a more realistic evaluation of the adjustment that needed to be applied to your measurements. If a datum is selected, the data is also not yet reduced to grid which could contribute to slightly large misclosures.

15.8 1D least squares messages

NO BENCHMARKS FOUND - THUS NO LEVELING ANALYSIS

You need a starting elevation to generate other elevations. This message could be due to a station naming problem of a benchmark.

NO ELEVATION FOR STATION A23 GENERATED

Missing data or station naming problems may create data which is not connected to the rest of your survey. This could prevent determination of elevations. If you did not want an elevation on this station the message can be ignored.

```
 MISCLOSURE OF MULTIPLE ELEV. DIFFERENCE MEASUREMENTS
   STATIONS           MISCLOSURE
 NP1      - SP1             .047
 SP1      - FCI1            .081
 SP1      - TOM1            .010
 NP1      - TOM1            .017
 SP1      - TOM1            .048
 SP1      - BM1             .575
 NP1      - SP1            2.406
 NP1      - SP1            1.124
 NP1      - SP1            1.336
 SP1      - TOM1            .943
 END OF MISCLOSURE REPORT
```

In the .1D (least squares adjustment report for elevations) any line that is shot multiple times is compared in this report. This includes lines shot in opposite directions or the same line shot on a reoccupation of a setup. It is very useful in looking for height of instrument or height of reflector errors. In this example NP1 - SP1 and SP1 - TOM1 are short lines and thus are indicating a problem. If NP1 - SP1 was a 6000 ft. line you would probably not be concerned. This report compares to the existing averaged elevation change, while the elev. diff. report in .GEN compares to the first measurement of that line.

```
 BENCHMARK ELEVATION RESIDUALS

   STATION       INPUT ELEV.     ADJUSTED ELEV.     ERROR EST.     RESIDUAL
```

B22	211.240	211.240	.001	.000
B6	203.670	203.670	.001	.000
B21	169.940	169.940	.001	.000

You should make sure that all of your benchmarks appear in the output file or something wrong occurred during their data entry. If you want the benchmarks not to adjust, their residuals (amount of adjustment) should be zero which is obtained by assigning them very small error estimates such as 0.001 ft. If you did want benchmark elevations to adjust based on a reasonable error estimate, ensure that the residual is of similar or smaller magnitude than its assigned error estimate. If the residual is dramatically larger than its error estimate (three times bigger is often used) it is important to check its assigned elevation for correctness.

RESIDUALS

FROM	TO	MEASURED	RESIDUAL		EST. ERROR
NP1	BM1	4.918	.011	(.3)	.037
NP1	BM2	6.170	.011	(.4)	.027
NP1	FCI1	62.304	-.197	(1.3)	.147
NP1	SP1	6.270	-.015	(.8)	.021
NP1	TOM1	-27.312	.004	(.3)	.014
SP1	BM1	-1.325	-.001	(.1)	.014
SP1	FCI1	55.729	.123	(1.1)	.116
SP1	TOM1	-33.557	-.005	(.3)	.015
SP1	BM2	-.056	-.019	(.5)	.035

ELEV. DIFF. RMS ERROR = .078 SNOOP RMS = .7
MAX. ELEV. DIFF. RESIDUAL NP1 - FCI1 OF .197

This listing shows how much adjustment (residual) had to be applied to the measured elevation differences. The residual should be of similar or smaller magnitude compared to the error estimate. If not, one should check the measurements which resulted in the elevation difference for correctness (incorrect station name, height of instrument, and height of reflector are common field data entry problems). One should also make sure the error estimates are of reasonable magnitude, and if not change default settings regarding error estimation and reprocess your data. See the beginning of this document if one has questions on assigning error estimates.

The number in parenthesis next to the residual is the snoop number which is equal to the absolute value of the residual divided by the error estimate. A snoop number larger than 3.0 is flagged with asterisks because it is indicative of a problem - the residual is more than three times its error estimate. Note 33% of your data (from a normal distribution/ bell shaped curve) will have snoop numbers greater than one so do not use a snoop number of greater than one as indicative of a problem (3.0 is more standard and implies 95 % confidence).

The distance from SP1 to FCI1 is over 4000 ft. which is why it has a large error estimate for its trig. leveling derived elevation change.

The maximum residual is highlighted at the end of the residual report. Also highlighted is the root-mean-square (average) residual and snoop number.

15.9 2D least squares messages

*** MAXIMUM NUMBER OF ITERATIONS REACHED ***

Least squares analysis of horizontal data requires preliminary coordinates (generated by the first processing program) and then the solution iterates in updating the coordinates to their least squares values. The solution usually quits iterations (called irritations by some users) when the updates are less than 0.005 ft.

*** SOLUTION DIVERGING - ITERATION HALTS ***

If blunders exist the updates in the horizontal least squares analysis can get bigger instead of smaller. This is called divergence, and means something is drastically wrong with your data. Always review results from the first processing program before continuing.

NO SUITABLE INFORMATION FOUND IN ELEVATION FILE
SO INPUT A MEAN PROJECT ELEVATION

If you have performed a horizontal survey (horizontal distances instead of slope distances/zenith angles), you desire to reduce measurements to state plane. If no benchmarks exist you need to input an average project elevation so reduction to grid due to elevation can be correctly performed.

ALL MEASUREMENTS ARE REDUCED TO THE NAD 83
0903 FLORIDA NORTH LAMBERT

COORDINATE AND DISTANCE UNITS ARE U.S. SURVEY FEET

If you have selected a datum make sure it is correct. In NAD 27 only the U.S. Survey Foot is allowed as a distance unit. In NAD 83 you are allowed meters, U.S. Survey Foot, or the International Foot. Control coordinates, distances, and elevation differences must be in the same units.
Make sure in NAD 83 the correct distance and coordinate units exist.

95% CONFIDENCE F STATISTIC STANDARD ERROR MULTIPLIER FOR 6 D.F. IS 3.21

To obtain standard deviations of final coordinates and error ellipses at 95% confidence a multiplier is applied which is a function of your degrees of freedom (level of redundancy). A more redundant survey (more checks) results in a lower multiplier because the greater number of checks enables you to be more comfortable with your final results.

			STANDARD ERRORS		ERROR ELLIPSE INFO.		
STATION	ADJUSTED X	ADJUSTED Y	IN X	IN Y	SU	SV	T
B22	626264.633	158284.560	.146	.066	.149	.059	78.2
A25	624980.787	158016.019	.011	.013	.013	.011	.0

The coordinate standard errors indicate if you performed the survey again under the same conditions you would be 95% confident of being within that amount of the least squares adjusted coordinate. The error ellipse defines an area centered about the adjusted coordinates where you would be 95% confident that performing the survey over under the same conditions would produce a coordinate within that area. SU and SV are the semi-major and semi-minor axes of the ellipse respectively, and T is the angle in decimal degrees (positive clockwise) from north of the semi-major axis.

DISTANCES

OCCUPIED STATION	SIGHTED STATION	DISTANCE	RESIDUAL		EST. ERROR
NP1	BM1	695.330	.014	(.6)	.023
NP1	BM2	236.275	.079	(1.3)	.062
NP1	FCI1	4203.195	.000	(.0)	.035
NP1	SP1	511.210	.005	(.6)	.008
NP1	TOM1	334.897	-.001	(.1)	.007
SP1	BM1	184.639	-.002	(.2)	.011
SP1	FCI1	3691.994	.002	(.1)	.019
SP1	TOM1	504.641	-.005	(.7)	.006
SP1	BM2	281.005	.014	(.6)	.022

DISTANCE RMS ERROR = .027 SNOOP RMS = .6
MAX. DISTANCE RESIDUAL NP1 - BM2 OF .079

Similar output exists for control coordinates, horizontal angles, and azimuths. Output measurement values are in their grid equivalents if a datum and state plane zone has been selected. This listing shows how much adjustment (residual) had to be applied to the measured distances, error estimates, and snoop numbers in parenthesis. The residual should be of similar or smaller magnitude when compared to the error estimate (snoop number less than 3.0). If not one should check the raw measurements which resulted in this abstracted (averaged) measurement, especially for hand entry items such as station name.

One should also make sure the error estimates are of reasonable magnitude, and if not change default settings regarding error estimation and reprocess your data. See the beginning of this document if one has questions on assigning error estimates.

```
----------------------------
     STANDARD ERROR OF UNIT WEIGHT IS      3.551
     WITH    6 DEGREES OF FREEDOM
 CHI SQUARED TEST ON ANALYSIS
   .454 <  3.551 <  1.449
   (LOW)            (HIGH)
 DOES NOT PASS AT THE 5 % SIGNIFICANCE LEVEL
```

This is a statistical test that your error estimates are reasonable. Do not be too alarmed if it does not pass because being 95% confident of measurement data is difficult to obtain. Making sure all of your residuals are reasonable in magnitude is more important than any statistical test.

```
----------------------------
 TRAVERSE CLOSURES NOT POSSIBLE TO COMPUTE
```

The angles and distances are measured in a way that does not allow the program to compute traverse closures. If your survey is supposed to satisfy Minimum Technical Standards of Florida you may want to add some measurements which make computation of closure possible.

```
----------------------------
 TRAVERSE CLOSURE REPORT
 SUM OF DISTANCES ALONG TRAVERSE IS        839.503
 CLOSURE IN X =     .010 CLOSURE IN Y =    -.024
 ANGULAR CLOSURE =    -9.0 SECONDS
 LINEAR ERROR OF CLOSURE (AFTER ROTATION) IS         .024
 BEFORE ROTATION PRECISION IS 1/       32550.
 AFTER  ROTATION PRECISION IS 1/       35381.
```

This is the report of a traverse closure. Links between adjusted coordinates are computed (not loops). Traversing starts on the inverse bearing between adjusted coordinates of the first leg. Coordinates are computed based on measured angles and distances (raw) until the terminating station is reached. Closure in X and Y is a comparison between adjusted and computed coordinates. Angular closure is based on comparing the computed bearing of the last leg to the bearing derived from inversing adjusted coordinates. To eliminate rotation error of the bearing of the first leg, the after rotation error linear error of closure compares the computed distance inverse between end stations to the distance inverse based on adjusted coordinates.

The after rotation precision is better than the before rotation precision because it uses the compared distance inverses, which is independent of the initial direction of the first leg. Measurements are reduced to grid in all computations if a state plane zone has been selected.

TOTAL LENGTH OF EVALUATED TRAVERSE DISTANCE =   .553 MILES

PRECISION BASED ON LATITUDE AND DEPARTURE CLOSURES = 1 /   3331.

PRECISION AFTER ORIENTATION CORRECTION = 1 /   3623.

This is an evaluation which is based on summing all linear errors of closure and sums of all individual traverse closure distances. This provides an overall 1/X if someone asks you for it.

15.10 Horizontal sideshot and .xyz/.soe generation messages

*** COORDINATES UNDETERMINED FOR STATION A23

If A23 was part of differential leveling this warning can be ignored. You may also not be interested in A23's coordinates. Normally this indicates a station naming problem or a lack of data.

STATION A23 IN Z DATA BUT NOT IN X,Y DATA

If A23 was part of differential leveling this warning can be ignored. You may also not be interested in A23's coordinates. Normally this indicates a station naming problem or a lack of data.

STATION A23 IN GRAPH DATA BUT NOT IN X,Y,Z DATA - IGNORED

A23 has feature code and other attribute information but no coordinates. All setups in differential leveling result in this warning which can be ignored.

STATION A23 IN GRAPH DATA BUT NO SOE INFO - IGNORED

A23 has feature code and other attribute information but no station-offset data.

Chapter 16. A general strategy for survey data processing using EFBP and storage of projects and executables

Experienced users of EFB and EFBP all have their own special techniques on how to most efficiently process data. This short synopsis is just one person's opinion on how to do it but it works!

(1) Create a working directory called PROC on the hard drive. Retain required EFBP files DEFAULT.SD and DEFAULT.CON in that directory. Also have metric and English equivalents of DEFAULT.SD and DEFAULT.CON called DEFAULT.SM / DEFAULT.CM (metric) and DEFAULT.SF / DEFAULT.CF (English) which get copied into .SD/.CON when units of a new job change from the previous job.

(2) Download field files into this directory.

(3) Convert .raw to .obs using program TSMTOASC and if not feeling too lazy quickly look at it in a text editor for any obvious problems or any important comments from the field. If feeling lazy do not look at .OBS and leave it up to EFBP to find the problems.

(4) Use program CTL to build a control file. If at all possible the control is in an existing .CTL, .XYZ, or ascii file. Import this data without having to type in any coordinates as that type of input (typing) is very prone to blunders.

(5) Run EFBP in first pass mode where you are only processing step 1 - abstracting and preliminary coordinate generation - the .GEN report. Import a correct DEFAULT.SD/.CON if necessary and make sure reasonable error estimate parameters are defined. Carefully define maximum spread tolerances above which tolerance values are asterisked (*).

(6) Review the .GEN report by looking for asterisks in my text editor search mode. If one finds a problem open up the .obs file in split screen mode and try to eliminate any problems. If problems were found go to (8). If no problems were found go to (9).

(7) Re-process only as far as the .GEN report and return to (6).

(8) Re-process to final .XYZ (and possibly .SOE) in the hopes that the .GEN report indicated all problems.

(9) Look at the vertical least squares .1D report. Do not look closely at elevation difference comparisons because that already was reported on in .GEN . View the snoop numbers of all elevation difference residuals and see if any problems exist. Problems are usually those above 3.0 though if the snoop number rms is high (above 1.5) also make sure the residuals are not systematic (all plus or minus in nature). Make sure the standard error of unit weight is reasonable.

If things are reasonable you are finished evaluating the .1d report and go to (12). If things are not reasonable it is probably a benchmark problem and go to (10). Note measuring to a point which was not really the benchmark is the same as a bad benchmark elevation.

(10) Run CTL and place non-fixed error estimates on all benchmarks. Suitable values would be 0.10 ft. or. 0.03 m. Reprocess only as far as .1d but turn on robustness using the EFBP menu. It is recommended to robust twice in succession.

(11) Review the robusted .1D results and hopefully find the problem(s). Re-run CTL, fix any problems, and re-set benchmark elevation error estimates to fixed (0.001). Return to (8).

(12) Look at the horizontal least squares .2D report. View the snoop numbers of all measurement residuals and see if any problems exist. Problems are usually those above 3.0 though if the snoop number rms is high (above 1.5) also make sure the residuals are not systematic (all plus or minus in nature). Make sure the standard error of unit weight is reasonable.

If things are reasonable you are finished evaluating the .2d report and go to (16). If things are not reasonable it is probably a horizontal control problem and go to (13). Note measuring to a point which was not really the control is the same as a bad control coordinate.

(13) Run CTL and place non-fixed error estimates on all horizontal control. Suitable values would be 0.10 ft. or. 0.03 m. Reprocess only as far as .2d but turn on robustness using the EFBP menu. Usually robust twice in succession for best results.

(14) Review the robusted .2D results and hopefully find the problem(s). Re-run CTL, fix any problems, and re-set horizontal control error estimates to fixed (0.001). Re-process to .XYZ and return to (12).

(15) You have acceptable .gen, .1d, and .2d reports. Review .XYZ and/or .SOE to ensure all stations have the required coordinate information (though the last part of processing usually indicates if a problem exists).

(16) If desired (not necessary) copy .XYZ, .SOE, chain, and tape files to a directory where all subsequent surveying/engineering calculations will occur.

Chapter 17. Description of executables and data files for EFBP

EFBP requires the following files in the directory in which data is being processed.

DEFAULT.CON - EFBP processing parameters
DEFAULT.SD - EFBP processing parameters

EFBP will assign reasonable default information in the DEFAULT files if they do not exist.

Instead of storing these files in the directory where data processing is going on, you can use the DOS Append command to store the files in a directory defined by Append. This will allow access to those files from any directory.

The following executables are part of the EFBP processing system. They need to be in the directory in which you are working or a path to the directory in which they reside must be defined.

EFBP.EXE
CTL.EXE
LOOK.EXE
CHK.EXE
LCHK.EXE
FIXIT.EXE

EFBP is a protected mode program so it will not run on a 286 or 8088 PC. It requires what is called an extender which are one of two files:

DOSXMSF.EXE - if running EFBP from a DOS command line
DOSXNT.EXE - if running EFBP from Windows

These two .exe files do not need to be "run", but only need to be in the directory in which you are working or a path must exist to the directory in which these files reside.

Data or report files generated or created in the field or office all have the project name followed by an extension which defines what it is.

Field files
.chn - chains (binary)
.cpx - chain name prefixes and largest suffixes (ascii)
.pre - station (point) name prefixes and largest suffixes (ascii)
.raw - survey measurements (binary)
.tap - taping (binary)

Office files (all ascii)

.obs - survey measurements

```
.ctl - control
.gen - abstracting and initial coordinate generation report
.1d  - 1D least squares report
.2d  - 2D least squares report
.xyz - final coordinate and point attribute file
.soe - final station, offset, elevation, attribute file
.lsa - redundant abstracted 2D survey data
.2sd - redundant abstracted 2D survey data error estimates
.cor - final redundant coordinates, scale factors, convergence angles
.geo - final redundant latitude & longitude
.red - graphics file of redundant data
```

Chapter 18. Example data and output

18.1 Description

The redundant portion of a job (BRIDGE) has been extracted from a larger job to illustrate the numerical processing of EFBP. Note the .obs file only reflects the total station measurement data (no taping, chains, prefixes, etc.) as that is all that is intended to be presented here.

18.2 Control file BRIDGE.CTL

```
G 00 BRIDGE
G 01 FCI1     55-72-A16                                      0903 27 29
G 02  2086126.879   524842.335                .100     .100
G 01 NP1      55-84-19A                                      0903 27 29
G 02  2086150.710   529045.338                .100     .100
G 01 BM1      55-84-A09                                      0903 27 29
G 02                          96.231                            .010
```

18.3 Observation file BRIDGE.OBS

```
H 00 319-RR-BRIDGE-REHAB     13:19:02 11/23/88  13:19:02 11/23/88
H 99 THIS IS A PRODUCTION PROJECT FOR STRUCTURES, AT SR319 SOUTH OF US 90 E ON
H 99 CAPITOL CIRCLE.
C 00 13:22:15 11/23/88  70 30.0 122222 BMD TLH WGC DOT
C 01 TOPCON     GTS-3B     Q31296     02 05 100
S 00 NP1      55-84-19A        P G1
S 01 14:18:00 11/23/88  5.400
O 00 BM1      55-84-A09        P G1
O 01 14:25:21    5.000          1 D 358 30  7.0  89 37 49     695.400 S
O 01 14:31:15    5.000          1 R 178 29 59.0 270 22 37     695.350 S
O 01 14:37:31    5.000          2 R  82 41 14.0 270 22 31     695.310 S
O 01 14:46:01    5.000          2 D 262 41 18.0  89 37 52     695.300 S
O 01 14:53:05    5.000          4 D 161 21 17.0  89 37 52     695.340 S
O 01 14:57:04    5.000          4 R 341 21 39.0 270 22 33     695.390 S
O 01 15:07:17    5.000          5 D  69 25  2.0  89 37 53     695.300 S
O 01 15:02:29    5.000          5 R 249 25 22.0 270 22 27     695.370 S
O 00 BM2                       P G1
O 01 15:13:10    5.000          5 R 260 16 44.0 271 24  7     236.400 S
O 01 15:11:36    5.000          5 D  80 16 54.0  88 36 15     236.290 S
O 00 FCI1     55-17-A16        P G1
O 01 14:33:48    5.850          1 D 359 59 59.0  89  8 51    4203.670 S
O 01 14:34:37    5.850          1 R 179 59 56.0 270 51 22    4203.660 S
O 01 14:50:04    5.850          4 D 162 50 54.0  89  8 42    4203.680 S
O 01 14:59:26    5.850          4 R 342 51  8.0 270 51 28    4203.630 S
O 01 15:00:39    5.850          5 R 250 54 54.0 270 51 23    4203.670 S
O 01 15:10:03    5.850          5 D  70 54 51.0  89  8 44    4203.670 S
O 00 SP1                       P G1
O 01 14:26:55    3.430          1 D 359 50 52.0  89 31 30     511.230 S
O 01 14:36:18    3.430          1 R 179 50 36.0 270 28 55     511.230 S
O 01 14:51:38    3.430          4 D 162 41 51.0  89 31 18     511.230 S
O 01 14:57:58    3.430          4 R 342 41 42.0 270 28 57     511.230 S
O 01 15:01:30    3.430          5 R 250 45 28.0 270 28 49     511.230 S
O 01 15:08:29    3.430          5 D  70 45 34.0  89 31 21     511.230 S
O 00 TOM1                      P G1
O 01 14:29:37    5.360          1 R 110  9  5.0 265 19 48     336.010 S
O 01 14:28:15    5.360          1 D 290  9  2.0  94 40 24     336.010 S
O 01 14:43:01    5.360          2 D 194 19 49.0  94 40 26     336.020 S
O 01 14:41:06    5.360          2 R  14 19 44.0 265 19 58     336.020 S
O 01 14:55:52    5.360          4 R 273  0  9.0 265 19 58     336.020 S
O 01 14:54:53    5.360          4 D  93  0  5.0  94 40 27     336.020 S
O 01 15:04:26    5.360          5 R 181  4  4.0 265 19 55     336.020 S
O 01 15:05:57    5.360          5 D   1  3 55.0  94 40 21     336.010 S
S 00 SP1                       P G1
S 01 16:21:59 11/23/88  3.680
O 00 BM1      55-84-A09        P G1
O 01 16:32:52    5.000          1 R 354 45 13.0 269 59 39     184.640 S
O 01 16:33:53    5.000          1 D 174 45 46.0  90  0 28     184.640 S
O 01 16:42:43    5.000          2 R  88 24 41.0 269 59 46     184.630 S
O 01 16:41:28    5.000          2 D 268 24 17.0  90  0 27     184.630 S
O 01 16:48:12    5.000          3 D 177 15  0.0  90  0 27     184.620 S
O 01 16:47:29    5.000          3 R 357 14 35.0 269 59 42     184.660 S
O 01 16:53:10    5.000          4 R 253  4 35.0 269 59 44     184.640 S
O 01 16:52:35    5.000          4 D  73  4 49.0  90  0 26     184.630 S
O 00 FCI1     55-72-A16        P G1
O 01 16:24:15    5.850          1 R   0  0  1.0 270 53 49    3692.450 S
```

```
O 01 16:38:03    5.850          1 D 179 59 44.0  89  6  4    3692.450 S
O 00 NP1      55-84-19A       P G1
O 01 16:35:39    5.140          1 D 359 49 31.0  90 32 39     511.230 S
O 01 16:25:39    5.140          1 R 179 49 24.0 269 27 38     511.220 S
O 01 16:44:59    5.140          2 R 273 28 55.0 269 27 36     511.230 S
O 01 16:40:04    5.140          2 D  93 28 48.0  90 32 35     511.230 S
O 01 16:49:36    5.140          3 D   2 19  4.0  90 32 38     511.230 S
O 01 16:46:22    5.140          3 R 182 18 59.0 269 27 34     511.230 S
O 01 16:50:54    5.140          4 D 258  8 31.0  90 32 38     511.230 S
O 01 16:54:45    5.140          4 R  78  8 54.0 269 27 43     511.230 S
O 00 TOM1                     P G1
O 01 16:34:51    5.360          1 D  38 19  6.0  93 37  5     505.650 S
O 01 16:28:43    5.360          1 R 218 18 59.0 266 23  0     505.650 S
O 01 16:40:40    5.360          2 D 131 58 14.0  93 37  6     505.640 S
O 01 16:43:37    5.360          2 R 311 58 16.0 266 23  2     505.650 S
O 01 16:46:58    5.360          3 R 220 48 26.0 266 23  5     505.650 S
O 01 16:48:54    5.360          3 D  40 48 22.0  93 37 13     505.650 S
O 01 16:51:30    5.360          4 D 296 38  0.0  93 37 12     505.650 S
O 01 16:53:41    5.360          4 R 116 38 22.0 266 23  6     505.650 S
S 00 SP1                      P G1
S 01 16:59:17 11/23/88  3.680
O 00 FCI1     55-72-A16       P G1
O 01 17:01:02    5.850          1 R 258 19 27.0 270 54 12    3692.440 S
O 01 17:03:38    5.850          1 D  78 19 28.0  89  6 16    3692.450 S
O 01 17:04:55    5.850          2 D 344 10 10.0  89  6  0    3692.450 S
O 01 17:07:34    5.850          2 R 164 10  6.0 270 53 52    3692.440 S
O 00 TOM1                     P G1
O 01 17:02:55    5.360          1 D 296 38 30.0  93 37 10     505.640 S
O 01 17:02:13    5.360          1 R 116 38 27.0 266 23 10     505.640 S
O 01 17:05:38    5.360          2 D 202 29 12.0  93 37  4     505.650 S
O 01 17:06:20    5.360          2 R  22 29  8.0 266 23  8     505.650 S
S 00 TOM1                     P G1
S 01 17:45:58 11/23/88  5.360
O 00 NP1      55-84-19A       P G1
O 01 18:01:42    5.140          1 D  71 48  3.0  85 22 37     335.980 S
O 01 18:02:29    5.140          1 R 251 47 50.0 274 37 32     335.990 S
O 01 18:16:04    5.140          2 D 318 47 51.0  85 22 42     335.980 S
O 01 18:10:09    5.140          2 R 138 47 42.0 274 37 27     336.000 S
O 01 18:21:54    5.140          3 R  46 15  9.0 274 37 30     335.980 S
O 01 18:16:54    5.140          3 D 226 15  7.0  85 22 40     335.990 S
O 01 18:23:17    5.140          4 R 303 57 51.0 274 37 27     335.990 S
O 01 18:34:40    5.140          4 D 123 57 58.0  85 22 39     335.990 S
OD01 18:38:34    5.140          4 R 303 57 59.0 274 37 34     335.990 S
O 00 SP1                      P G1
O 01 18:04:38    3.450          1 R 179 58 59.0 273 35 10     505.630 S
O 01 17:52:33    3.450          1 D 359 59 58.0  86 24 56     505.630 S
O 01 18:11:27    3.450          2 R  66 58 53.0 273 35 13     505.630 S
O 01 18:12:43    3.450          2 D 246 58 58.0  86 25  2     505.640 S
O 01 18:20:02    3.450          3 R 334 26 19.0 273 35 15     505.640 S
O 01 18:19:17    3.450          3 D 154 26 15.0  86 25  0     505.630 S
O 01 18:25:47    3.450          4 R 232  9  0.0 273 35 13     505.640 S
O 01 18:32:01    3.450          4 D  52  9 10.0  86 24 55     505.630 S
OD01 18:45:18    3.450          4 R 232  9 33.0 273 35 12     505.640 S
O 00 TBM1                     P G1
O 01 17:59:39    0.760          1 D  38 30 28.0  89 38 40     330.010 S
O 01 18:03:11    0.760          1 R 218 30 26.0 270 21 31     330.010 S
O 01 18:13:21    0.760          2 D 285 30 26.0  89 38 42     330.020 S
O 01 18:08:31    0.760          2 R 105 30 20.0 270 21 24     330.040 S
O 01 18:18:30    0.760          3 D 192 57 43.0  89 38 45     330.010 S
O 01 18:20:35    0.760          3 R  12 57 49.0 270 21 27     330.020 S
O 01 18:42:51    0.760          4 R 270 39 48.0 270 21 24     329.930 S
O 01 18:33:21    0.760          4 D  90 40 36.0  89 38 38     330.050 S
OD01 18:25:03    0.760          4 R 270 40 25.0 270 21 29     330.020 S
O 00 WP1                      P G1
O 01 18:05:53    5.220          1 R 222  1 58.0 270 18 42     500.790 S
O 01 17:58:24    5.220          1 D  42  1 51.0  89 41 24     500.780 S
O 01 18:07:17    5.220          2 R 109  1 48.0 270 18 43     500.790 S
O 01 18:14:47    5.220          2 D 289  1 52.0  89 41 27     500.790 S
O 01 18:21:14    5.220          3 R  16 29  6.0 270 18 45     500.790 S
O 01 18:17:28    5.220          3 D 196 29  5.0  89 41 32     500.790 S
O 01 18:23:54    5.220          4 R 274 11 54.0 270 18 49     500.790 S
O 01 18:33:57    5.220          4 D  94 12 13.0  89 41 34     500.790 S
OD01 18:41:18    5.220          4 R 274 12 26.0 270 18 45     500.800 S
C 00 14:20:55 11/29/88  50 30.2 000000 BMD BMD GWF TLH
C 01 TOPCON     GTS-3B     Q31296      2  5 100
S 00 SP1                      P G1
S 01 14:21:23 11/29/88  5.000
O 00 BM1      55-84-A09       P G1
O 01 14:25:28    5.000          1 D 359 58 58.0  90 24 25     184.630 S
O 01 14:26:49    5.000          1 R 179 59 18.0 269 36 14     184.680 S
O 00 BM2                      P F1
O 01 14:30:38    5.000          1 D 177  3 36.0  90  0 57     281.020 S
O 01 14:29:45    5.000          1 R 357  3 26.0 269 59 35     280.990 S
O 00 NP1      55-84-19A       P G1
O 01 14:33:27    5.000          1 D 185  4 15.0  90 42 24     511.230 S
O 01 14:34:42    5.000          1 R   5  4 12.0 269 18  3     511.280 S
```

18.4 Abstracting and preliminary traverse report BRIDGE.GEN

```
PROJECT BRIDGE   PARAMETERS
USE REPETITION ERRORS PLUS ADD-ONS IN ERROR ESTIMATION
COMPUTE COORDINATE STANDARD ERRORS AND ERROR ELLIPSES
CORRECT FOR EARTH CURVATURE AND ATMOSPHERIC REFRACTION
ROBUST ERROR ESTIMATE PROMPT WILL NOT APPEAR
PROCESS TO FINAL COORDINATE .XYZ FILE

FOLLOWING USED AS ADD-ONS TO ERROR FROM REPETITON
DISTANCE   DISTANCE     HORZ.     AZIMUTH   TRIG. LEV.   TRIG. LEV.  DIFF. LEV
CONSTANT     PPM     ANGLE (SEC)   (SEC)     CONSTANT       PPM      CONSTANT
     .010      5.00       6.0         10.0        .030       100.00        .010

FOLLOWING ARE USER DEFINED ERROR ESTIMATES
  DISTANCE   DISTANCE     HORZ.      AZIMUTH    TRIG. LEV.   DIFF. LEV.
  CONSTANT     PPM     ANGLE (SEC)    (SEC)      CONSTANT     CONSTANT
      .020       5.00        12.0       10.0          .050         .010

SETUP ERROR (ALWAYS USED) =   .005
FLAG MAXIMUM SPREADS ABOVE
(23) DISTANCE =   .030 (24) ANGLES =   20.0 (25) ELEV. DIFFERENCES =   .100


WARNING - CALIBRATION RECORD WITHOUT DATA

PROCESSING SETUP #   1 AT STATION NP1
REPETITION STANDARD ERRORS
SIGHTED      HORIZONTAL DISTANCE      ELEVATION DIFF.           COMPARE
STATION      SD     SD      MAX        SD     SD      MAX      HORZ.     ELEV.
                  (MEAN)  SPREAD            (MEAN)   SPREAD    DIST.     DIFF.
BM1        .040    .014 *  .055*    .044   .015     .056
BM2        .077    .055 *  .055*    .020   .014     .014
FCI1       .019    .008 *  .036*    .134   .055 *   .210*
SP1        .000    .000    .000     .026   .010     .038
TOM1       .006    .002    .007     .018   .006     .021
REPETITION STANDARD ERRORS FOR ANGLES
BS STATION     FS STATION     SD    SD (MEAN)  MAX SPREAD
BM1            BM2            21.       15.         15.
BM1            FCI1           12.        5.         14.
BM1            SP1            17.        7.     *   23.*
BM1            TOM1           13.        5.     *   22.*

PROCESSING SETUP #   1 AT STATION SP1
REPETITION STANDARD ERRORS
SIGHTED      HORIZONTAL DISTANCE      ELEVATION DIFF.           COMPARE
STATION      SD     SD      MAX        SD     SD      MAX      HORZ.     ELEV.
                  (MEAN)  SPREAD            (MEAN)   SPREAD    DIST.     DIFF.
BM1        .012    .004    .024     .005   .002     .007
FCI1       .001    .001    .001     .089   .063     .063
NP1        .003    .001    .009     .021   .008     .032      .006     -.036
TOM1       .004    .001    .009     .018   .006     .025
REPETITION STANDARD ERRORS FOR ANGLES
BS STATION     FS STATION     SD    SD (MEAN)  MAX SPREAD
BM1            FCI1           35.       25.     *   25.*
BM1            NP1            18.        6.     *   27.*
BM1            TOM1           17.        6.     *   25.*

PROCESSING SETUP #   2 AT STATION SP1
REPETITION STANDARD ERRORS
SIGHTED      HORIZONTAL DISTANCE      ELEVATION DIFF.           COMPARE
STATION      SD     SD      MAX        SD     SD      MAX      HORZ.     ELEV.
                  (MEAN)  SPREAD            (MEAN)   SPREAD    DIST.     DIFF.
FCI1       .008    .004    .009     .214   .107 *   .268*     .006      .080
TOM1       .006    .003    .007     .023   .012     .027      .003      .010
REPETITION STANDARD ERRORS FOR ANGLES
BS STATION     FS STATION     SD    SD (MEAN)  MAX SPREAD
FCI1           TOM1            1.        0.          2.

PROCESSING SETUP #   1 AT STATION TOM1
REPETITION STANDARD ERRORS
SIGHTED      HORIZONTAL DISTANCE      ELEVATION DIFF.           COMPARE
STATION      SD     SD      MAX        SD     SD      MAX      HORZ.     ELEV.
                  (MEAN)  SPREAD            (MEAN)   SPREAD    DIST.     DIFF.
NP1        .007    .002    .012     .008   .003     .012      .006     -.014
SP1        .005    .002    .008     .016   .006     .022     -.004     -.043
TBM1       .036    .013 *  .081*    .008   .003     .013
WP1        .004    .001    .009     .020   .007     .029
REPETITION STANDARD ERRORS FOR ANGLES
BS STATION     FS STATION     SD    SD (MEAN)  MAX SPREAD
NP1            SP1            16.        6.     *   40.*
NP1            TBM1           14.        5.     *   34.*
NP1            WP1             8.        3.         14.
WARNING - CALIBRATION RECORD WITHOUT DATA

PROCESSING SETUP #   3 AT STATION SP1
```

```
REPETITION STANDARD ERRORS
SIGHTED      HORIZONTAL DISTANCE      ELEVATION DIFF.           COMPARE
STATION      SD     SD     MAX        SD    SD      MAX      HORZ.     ELEV.
                  (MEAN)  SPREAD         (MEAN)   SPREAD     DIST.     DIFF.
BM1        .036    .025    .025     .024   .017     .017     -.014      .046
BM2        .021    .015    .015     .031   .022     .022
NP1        .036    .025    .025     .047   .033     .033     -.007     -.015
REPETITION STANDARD ERRORS FOR ANGLES
BS STATION     FS STATION     SD    SD (MEAN)  MAX SPREAD
BM1            BM2            21.      15.         15.
BM1            NP1            16.      11.         11.
    2 OF     8 STATIONS ARE HORIZONTAL SIDESHOTS

TRAVERSE CLOSURE REPORT
LINEAR ERROR OF CLOSURE IS         .114 FT.

PRECISION IS 1/       39844.


STATION                   X COOR.         Y COOR.

FCI1                  2086126.879      524842.335
SP1                   2086149.179      528534.161
TOM1                  2086464.440      528928.191
NP1                   2086150.710      529045.338

STATION  BM1      HAS COORDINATES GENERATED BY ANGLE AND DISTANCE
FROM KNOWN BACKSIGHT
X=  2086164.912 Y=   528350.153


STATION  BM2      HAS COORDINATES GENERATED BY ANGLE AND DISTANCE
FROM KNOWN BACKSIGHT
X=  2086110.941 Y=   528812.434


FINAL GENERATED COORDINATES

STATION                    X COOR.         Y COOR.

FCI1                   2086126.879      524842.335
NP1                    2086150.710      529045.338
BM1                    2086164.912      528350.153
BM2                    2086110.941      528812.434
SP1                    2086149.179      528534.161
TOM1                   2086464.440      528928.191

DISTANCE RESIDUALS

            DISTANCE                   RESIDUAL

            NP1      - BM1                 .000
            NP1      - BM2                 .000
            NP1      - FCI1                .124
            NP1      - SP1                 .032
            NP1      - TOM1                .009
            SP1      - BM1                -.040
            SP1      - FCI1                .100
            SP1      - TOM1                .014
            SP1      - BM2                 .117

ANGLE RESIDUALS

                 ANGLE                                 RESIDUAL (SEC)

                 BM1      - NP1      - BM2                  .0
                 BM1      - NP1      - FCI1                 .0
                 BM1      - NP1      - SP1                -4.9
                 BM1      - NP1      - TOM1               -1.0
                 BM1      - SP1      - FCI1               23.9
                 BM1      - SP1      - NP1                37.6
                 BM1      - SP1      - TOM1               35.5
                 FCI1     - SP1      - TOM1                 .0
                 NP1      - TOM1     - SP1                  .0
                 BM1      - SP1      - BM2                36.3
                 BM1      - SP1      - NP1                94.3
```

18.5 Vertical least squares report BRIDGE.1D

```
MISCLOSURE OF MULTIPLE ELEV. DIFFERENCE MEASUREMENTS
  STATIONS           MISCLOSURE
NP1      - SP1             .036
SP1      - FCI1            .080
SP1      - TOM1            .010
NP1      - TOM1            .014
SP1      - TOM1            .039
```

```
SP1      - BM1            .046
NP1      - SP1            .003
END OF MISCLOSURE REPORT

    2 OF     8 STATIONS IDENTIFIED AS VERTICAL SIDESHOTS
BAND IS   4 STATIONS
LEVEL NETWORK ADJUSTMENT

NUMBER OF BENCHMARKS =     1
NUMBER OF STATIONS =     6
NUMBER OF MEASUREMENTS =     9
NUMBER OF REQUIRED TERMS FOR NORMAL EQUATIONS =      26

RESULTS OF ADJUSTMENT

BENCHMARK ELEVATION RESIDUALS

  STATION       INPUT ELEV.     ADJUSTED ELEV.     ERROR EST.     RESIDUAL

  BM1               96.231            96.231         .010        .000  ( .0)

BENCHMARK RMS ERROR =       .000 SNOOP RMS =   .0
MAX. BENCHMARK RESIDUAL AT STATION         BM1      OF       .000


RESIDUALS

  FROM          TO            MEASURED      RESIDUAL      EST. ERROR

  NP1           BM1              4.928          .007  ( .1)    .115
  NP1           BM2              6.171          .012  ( .2)    .068
  NP1           FCI1            62.668        -.274  ( .5)    .505
  NP1           SP1              6.269        -.014  ( .2)    .058
  NP1           TOM1           -27.313          .005  ( .1)    .048
  SP1           BM1             -1.319        -.001  ( .0)    .040
  SP1           FCI1            56.013          .126  ( .4)    .341
  SP1           TOM1           -33.557        -.005  ( .1)    .052
  SP1           BM2              -.054        -.017  ( .2)    .080

ELEV. DIFF.  RMS ERROR =       .101 SNOOP RMS =   .3
MAX. ELEV. DIFF. RESIDUAL         NP1      -         FCI1     OF       .274


95% CONFIDENCE F STATISTIC STANDARD ERROR MULTIPLIER FOR    4 D.F. IS  3.73

   STATION        ADJUSTED ELEV.     STANDARD ERROR

   BM1                 96.231              .014
   NP1                 91.296              .075
   SP1                 97.551              .057
   BM2                 97.480              .097
   FCI1               153.690              .410
   TOM1                63.988              .079


    STANDARD ERROR OF UNIT WEIGHT IS       .384
     WITH    4 DEGREES OF FREEDOM

CHI SQUARED TEST ON ANALYSIS
  .348 <   .384 <  1.540
  (LOW)           (HIGH)
PASSES AT THE 5 % SIGNIFICANCE LEVEL
```

18.6 Horizontal least squares report BRIDGE.2D

```
    PARAMETRIC HORIZONTAL LEAST SQUARES ADJUSTMENT

ALL MEASUREMENTS ARE REDUCED TO THE NAD 27
0903 FLORIDA NORTH LAMBERT

COORDINATE AND DISTANCE UNITS ARE U.S. SURVEY FEET
BAND IS   4 STATIONS
NUMBER OF TERMS REQUIRED IN NORMAL EQUATIONS =     111

95% CONFIDENCE F STATISTIC STANDARD ERROR MULTIPLIER FOR   12 D.F. IS  2.77

                        RESULTS OF ADJUSTMENT

                                         STANDARD ERRORS  ERROR ELLIPSE INFO.
STATION            ADJUSTED X   ADJUSTED Y   IN X   IN Y     SU      SV     T

BM2               2086110.939  528812.487  .308   .240   .308   .240   88.0
NP1               2086150.710  529045.307  .318   .227   .318   .227  -89.7
SP1               2086149.164  528534.124  .286   .227   .286   .227  -89.8
BM1               2086164.926  528350.164  .278   .231   .278   .231  -89.1
```

```
FCI1             2086126.879   524842.366   .318   .227   .318   .227  -89.7
TOM1             2086464.428   528928.151   .311   .232   .313   .229  -80.8

                        RESIDUALS IN THE OBSERVATIONS

                          CONTROL POINT COORDINATES

 STATION           X RESIDUAL  X EST. ERROR  Y RESIDUAL  Y EST. ERROR

 FCI1               .000  ( .0)    .100      -.031  ( .3)    .100
 NP1                .000  ( .0)    .100       .031  ( .3)    .100

EASTING CONTROL RMS =      .000 SNOOP RMS =   .0
MAX. EASTING RESIDUAL AT FCI1     OF      .000
NORTHING CONTROL RMS =      .031 SNOOP RMS =   .3
MAX. NORTHING RESIDUAL AT FCI1     OF      .031


                            DISTANCES

   OCCUPIED            SIGHTED          DISTANCE    RESIDUAL    EST. ERROR
   STATION             STATION

   NP1                 BM1               695.300     .012  ( .4)     .027
   NP1                 BM2               236.264     .072  (1.1)     .066
   NP1                 FCI1             4203.004    -.004  ( .1)     .039
   NP1                 SP1               511.188     .004  ( .3)     .011
   NP1                 TOM1              334.883     .003  ( .3)     .010
   SP1                 BM1               184.632    -.002  ( .2)     .015
   SP1                 FCI1             3691.825     .000  ( .0)     .022
   SP1                 TOM1              504.620    -.007  ( .8)     .009
   SP1                 BM2               280.993     .018  ( .7)     .026

DISTANCE RMS ERROR =       .025 SNOOP RMS =   .6
MAX. DISTANCE RESIDUAL         NP1      -         BM2      OF       .072


                          ANGLES

   BACKSIGHT   OCCUPIED    FORESIGHT       ANGLE      RESIDUAL    EST. ERROR
   STATION     STATION     STATION                    (SECONDS)    (SECONDS)

   BM1         NP1         BM2            10-51-37.0   -17.9  ( .8)     21.5
   BM1         NP1         FCI1            1-29-42.7    -4.4  ( .4)     10.8
   BM1         NP1         SP1             1-20-26.2   -15.1  (1.1)     13.3
   BM1         NP1         TOM1          291-38-44.4   -13.1  (1.2)     11.3
   BM1         SP1         FCI1            5-14-23.0   -11.5  ( .4)     31.5
   BM1         SP1         NP1           185- 4- 8.7    -4.3  ( .3)     13.6
   BM1         SP1         TOM1          223-33-36.1    -1.9  ( .1)     13.3
   FCI1        SP1         TOM1          218-19- 1.5    -2.1  ( .3)      6.8
   NP1         TOM1        SP1           288-11-15.1     6.3  ( .5)     12.3
   BM1         SP1         BM2           177- 4-23.0   -18.1  ( .8)     22.0
   BM1         SP1         NP1           185- 5- 5.5    52.5  (2.8)     18.5

ANGLE RMS ERROR =       19.1 SECONDS  SNOOP RMS =  1.1
MAXIMUM ANGLE RESIDUAL         BM1      -         SP1      -         NP1
OF    52.5 SEC.

    STANDARD ERROR OF UNIT WEIGHT IS      1.148
    WITH   12 DEGREES OF FREEDOM
CHI SQUARED TEST ON ANALYSIS
  .606 <  1.148 <  1.324
  (LOW)            (HIGH)
PASSES AT THE 5 % SIGNIFICANCE LEVEL

---------------------------------------------------
TRAVERSE CLOSURE REPORT
SUM OF DISTANCES ALONG TRAVERSE IS      4531.328
CLOSURE IN X =     .016 CLOSURE IN Y =     .014
ANGULAR CLOSURE =     8.4 SECONDS
LINEAR ERROR OF CLOSURE (AFTER ROTATION) IS         .014
BEFORE ROTATION PRECISION IS 1/      210529.
AFTER  ROTATION PRECISION IS 1/      320339.

STATION         BEARING       DISTANCE        X            Y

FCI1                                    2086126.879   524842.366
SP1          N 0-20-45.1E     3691.825  2086149.164   528534.124
TOM1         N38-39-48.7E      504.627  2086464.428   528928.151
NP1          N69-31-20.1W      334.880  2086150.710   529045.307
```

TOTAL LENGTH OF EVALUATED TRAVERSE DISTANCE =   .858 MILES

PRECISION BASED ON LATITUDE AND DEPARTURE CLOSURES = 1 /   210529.

```
PRECISION AFTER ORIENTATION CORRECTION = 1 /    320339.
```

18.7 Output adjusted state plane coordinate file BRIDGE.COR

STATION	X COOR.	Y COOR.	SCALE FACTOR	CONVERGENCE
BM1	2086164.926	528350.164	.99996078	+ 0- 8- 14.7
BM2	2086110.939	528812.487	.99996089	+ 0- 8- 14.4
FCI1	2086126.879	524842.366	.99995996	+ 0- 8- 14.4
NP1	2086150.710	529045.307	.99996095	+ 0- 8- 14.6
SP1	2086149.164	528534.124	.99996083	+ 0- 8- 14.6
TOM1	2086464.428	528928.151	.99996092	+ 0- 8- 16.4

18.8 Output adjusted geodetic coordinate file BRIDGE.GEO

STATION	LATITUDE	LONGITUDE
BM1	30-27- 9.27887	84-13- 35.57970
BM2	30-27- 13.85650	84-13- 36.18382
FCI1	30-26- 34.55742	84-13- 36.11044
NP1	30-27- 16.16015	84-13- 35.72307
SP1	30-27- 11.10020	84-13- 35.75473
TOM1	30-27- 14.99302	84-13- 32.14206

18.9 Input redundant horizontal measurement file BRIDGE.LSA

```
FCI1          2086126.879       524842.335    0
NP1           2086150.710       529045.338    1
NP1               BM1                  695.330    0
NP1               BM2                  236.275    0
NP1               FCI1                4203.195    0
NP1               SP1                  511.211    0
NP1               TOM1                 334.897    0
SP1               BM1                  184.640    0
SP1               FCI1                3691.994    0
SP1               TOM1                 504.642    0
SP1               BM2                  281.005    1
BM1        NP1        BM2        10 51 37.00 0
BM1        NP1        FCI1        1 29 42.67 0
BM1        NP1        SP1         1 20 26.17 0
BM1        NP1        TOM1      291 38 44.37 0
BM1        SP1        FCI1        5 14 23.00 0
BM1        SP1        NP1       185  4  8.75 0
BM1        SP1        TOM1      223 33 36.13 0
FCI1       SP1        TOM1      218 19  1.50 0
NP1        TOM1       SP1       288 11 15.13 0
BM1        SP1        BM2       177  4 23.00 0
BM1        SP1        NP1       185  5  5.50 1
0              0                     0  0    .00   1
BM1                 2086164.912             528350.153    0
BM2                 2086110.941             528812.434    0
SP1                 2086149.179             528534.161    0
TOM1                2086464.440             528928.191    1
```

18.10 Input redundant horizontal measurement error estimate file BRIDGE.2SD

```
FCI1             .100         .100    0
NP1              .100         .100    1
NP1               BM1                     .027    0
NP1               BM2                     .066    0
NP1               FCI1                    .039    0
NP1               SP1                     .011    0
NP1               TOM1                    .010    0
SP1               BM1                     .015    0
SP1               FCI1                    .022    0
SP1               TOM1                    .009    0
SP1               BM2                     .026    1
BM1        NP1        BM2          21.00 0
BM1        NP1        FCI1         10.71 0
BM1        NP1        SP1          13.10 0
BM1        NP1        TOM1         10.75 0
BM1        SP1        FCI1         31.00 0
BM1        SP1        NP1          12.22 0
BM1        SP1        TOM1         11.92 0
FCI1       SP1        TOM1          6.50 0
```

```
NP1        TOM1       SP1         11.72 0
BM1        SP1        BM2         21.00 0
BM1        SP1        NP1         17.50 1
```

18.11 Final coordinate file BRIDGE.XYZ

```
G 00 BRIDGE.XYZ
G 01 FCI1     55-17-A16        P G1                            0903 27 29
G 02  2086126.879  524842.366   153.690     .318     .227     .410
G 01 NP1      55-84-19A        P G1                            0903 27 29
G 02  2086150.710  529045.307    91.296     .318     .227     .075
G 01 BM1      55-84-A09        P G1                            0903 27 29
G 02  2086164.926  528350.164    96.231     .278     .231     .014
G 01 BM2                       P G1                            0903 27 29
G 02  2086110.939  528812.487    97.480     .308     .240     .097
G 01 SP1                       P G1                            0903 27 29
G 02  2086149.164  528534.124    97.551     .286     .227     .057
G 01 TOM1                      P G1                            0903 27 29
G 02  2086464.428  528928.151    63.988     .311     .232     .079
G 01 TBM1                      P G1                            0903 27 29
G 02  2086142.655  528854.965    70.643
G 01 WP1                       P G1                            0903 27 29
G 02  2085970.233  528847.327    66.847
```